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The Elasto-Plastic Analysis of Two Experimental Portal Frames
by Professor J. W. Roderick (Member)

Some Points of Structural Interest at Calder Hall 'A' Nuclear Power Station
by W. S. Watts (Associate-Member)

The Design of the Unbraced Stabbogen Arch
Written Discussion on the Paper by Chin Fung Kee (Associate-Member)

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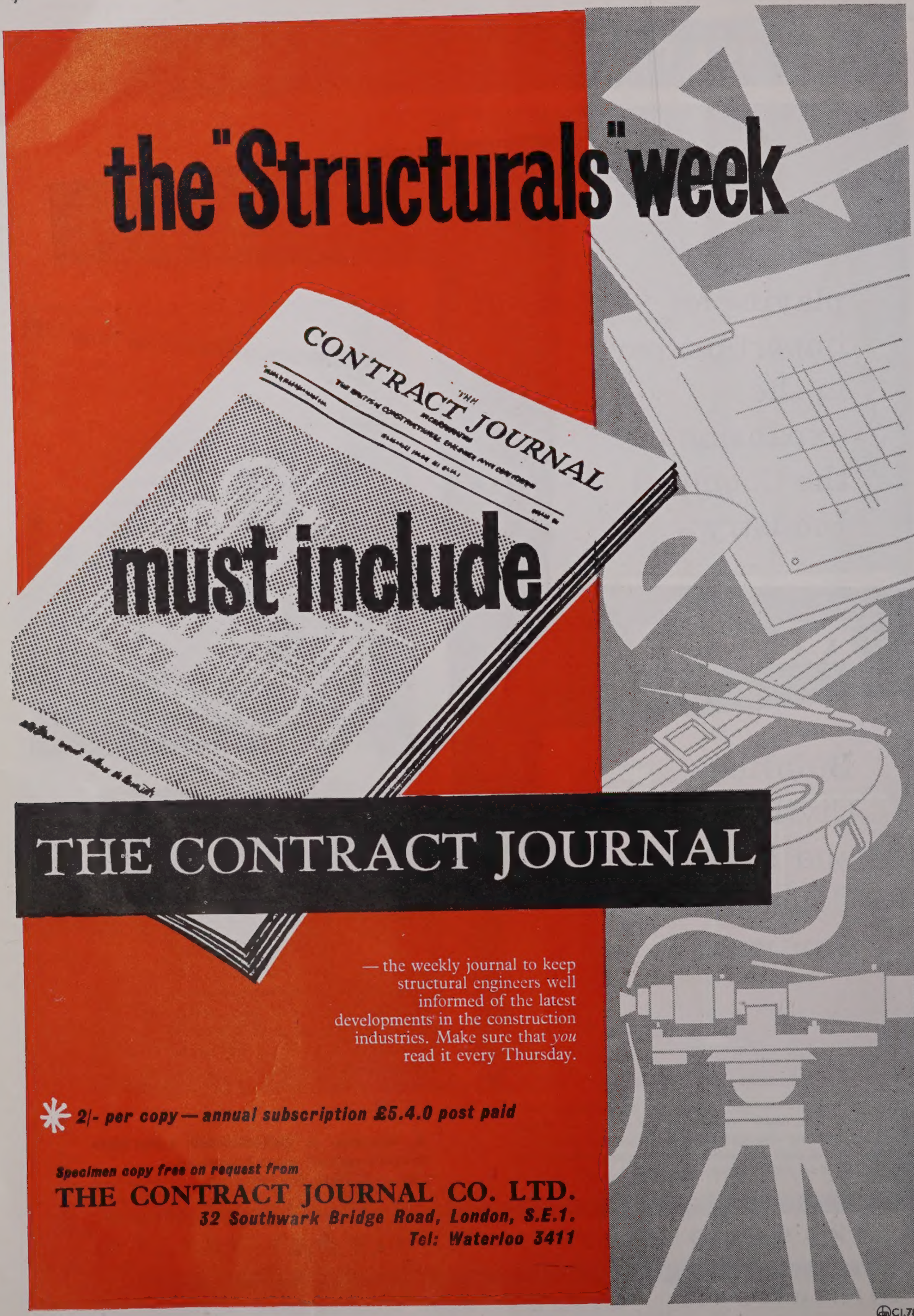
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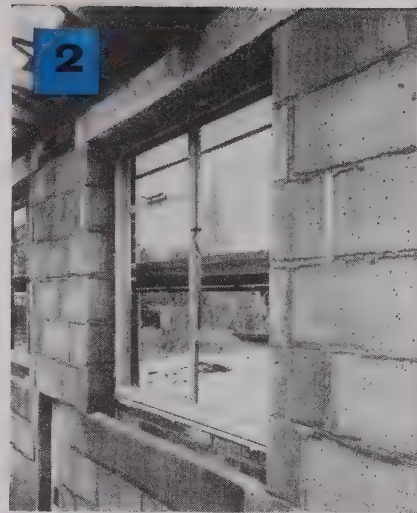

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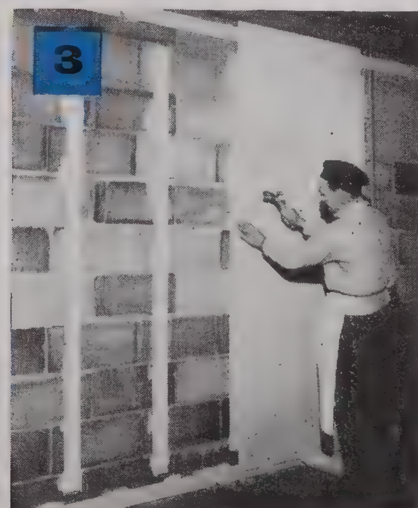

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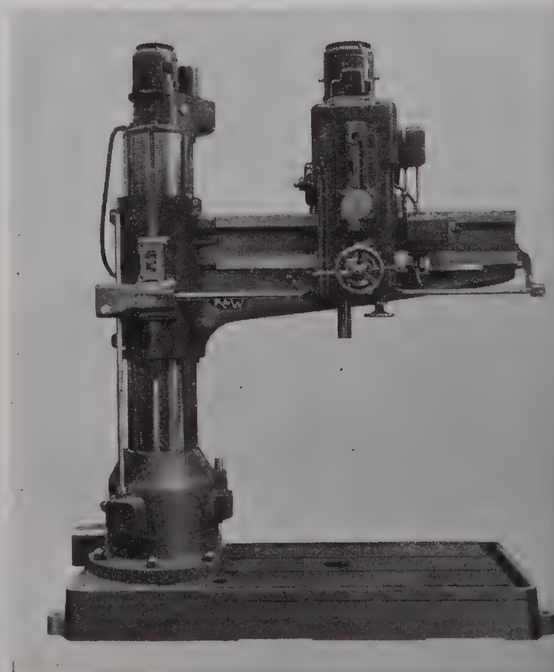


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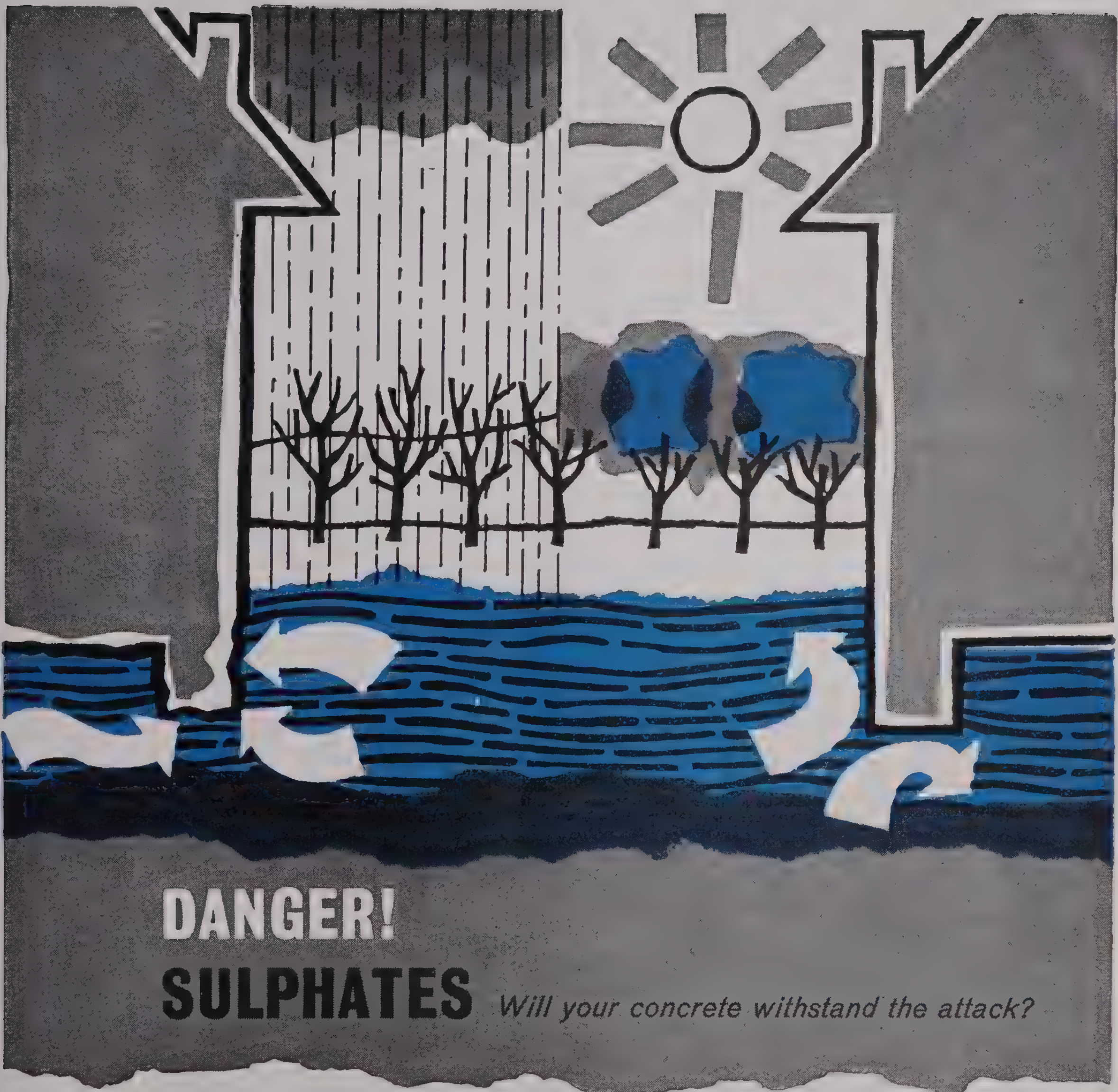
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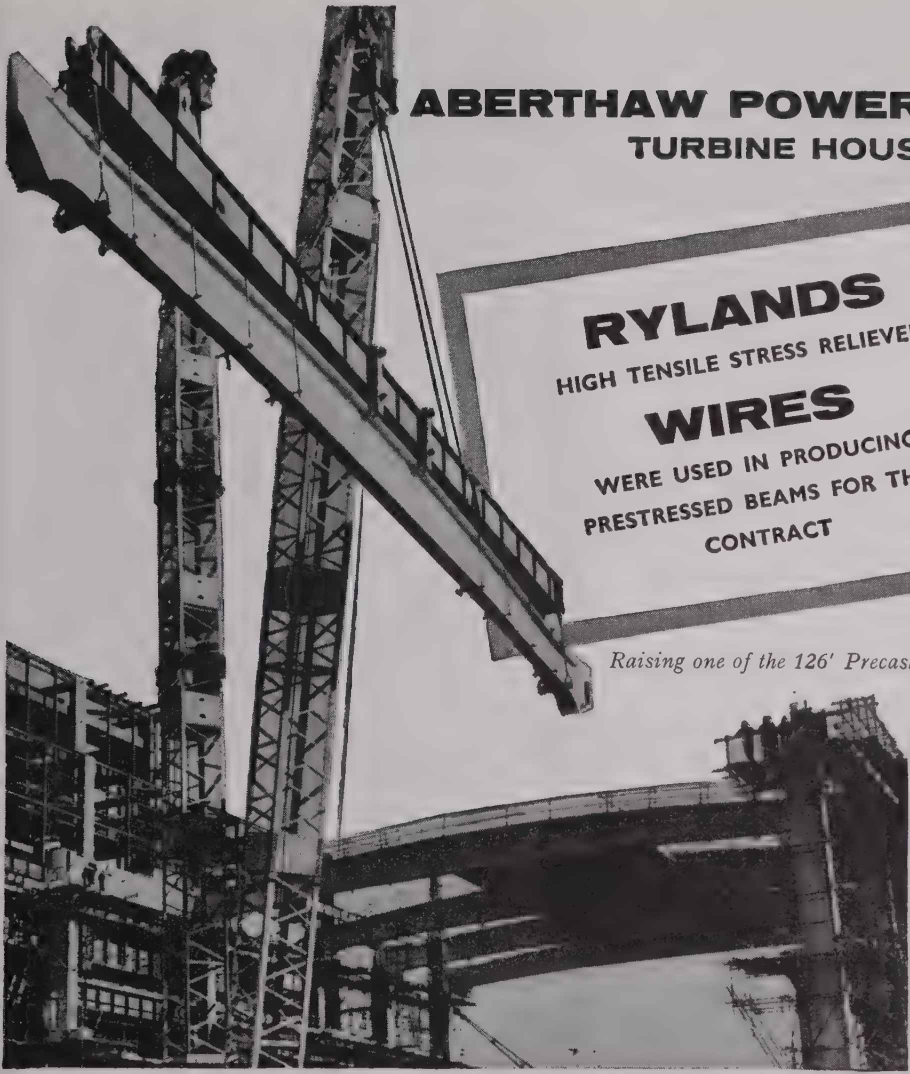

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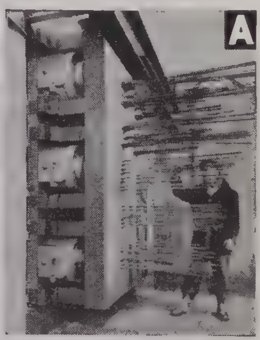

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CONTENTS

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The Elasto-Plastic Analysis of Two Experimental Portal Frames

by Professor J. W. Roderick

M.A., Ph.D., M.I.Struct.E., M.I.C.E.

1. Synopsis

Using a stress/strain curve closely representative of the true material properties, the moment/curvature relationship in the elasto-plastic range has been obtained for a batch of rolled steel joists used in the construction of two half-scale rectangular portal frames tested to destruction under combined vertical and horizontal loads. This information forms the basis for an accurate theoretical determination of bending moment distributions and deformations at or near the collapse load as determined by the simple plastic theory.

By taking account of the deformation moments associated with the side sway deflection, it is possible to show that the observed catastrophic collapse of the pinned base frame resulted from a condition of instability. For the fixed base frame these moments are of less consequence. It is demonstrated that strain hardening is then the predominant effect, and enables the frame to carry loads well in excess of the collapse load calculated according to the simple plastic theory, as was the case in the actual test.

2. Introduction

It is now well known that the simple plastic theory based on an idealized stress/strain relationship for mild steel enables a satisfactory estimate to be made of the load carrying capacity of a rigid frame structure which resists its load largely by bending. It does not, however, afford an accurate means of assessing deformations at or near this maximum load since the assumed stress/strain relationship neglects the occurrence in rolled sections, of a limit of proportionality well below the yield point, and more important still, takes no account of the development of strain hardening which in turn enables highly strained sections of a member to resist bending moments well in excess of the full plastic value as calculated by the simple theory. In the present paper an attempt has been made to obtain a more exact determination of deformations using as a basis a stress/strain relationship as nearly representative as possible of the true material properties. In this way it has been possible to study in detail both the influence of strain hardening and also the effect of overall deformation on the moment distribution in a given frame. This solution is of value, not so much as a means of assessing deformations, but for the light it throws on the behaviour of a rigid frame in the vicinity of the collapse load as determined from the simple plastic theory.

The analytical method has been checked against the results obtained from tests on two experimental rectangular portal frames carried out by Baker and Roderick and described in an earlier paper[1]. These frames were fabricated from 8 in. × 4 in. @ 18 lb. R.S.J. and each had a span of 16 ft. and a stanchion height of 8 ft. The first frame (FSF1A) was pinned at the bases and loaded as shown in Fig. 1; the second (FSF2A) was effectively fixed at the bases, being attached to a heavy compound girder, and was loaded as shown in Fig. 2. In each test, loading was continued until the frame either collapsed catastrophically or developed deflections sufficiently large to render it unserviceable.

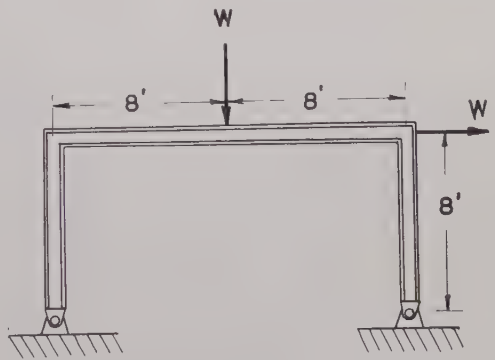

Fig. 1

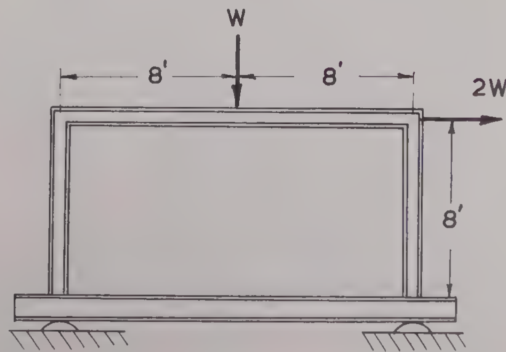

Fig. 2

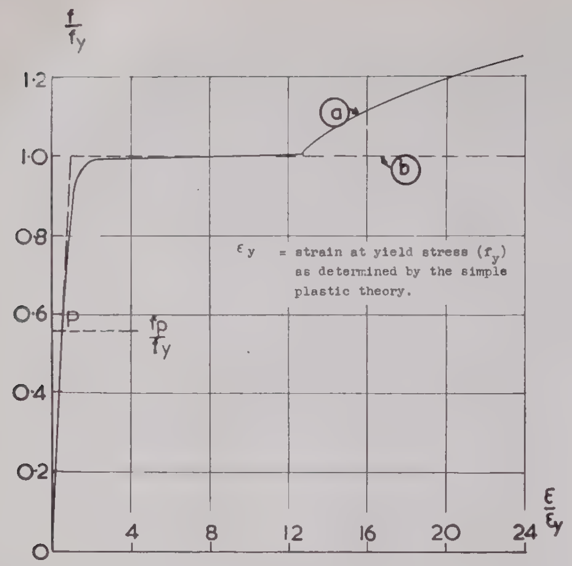

Fig. 3

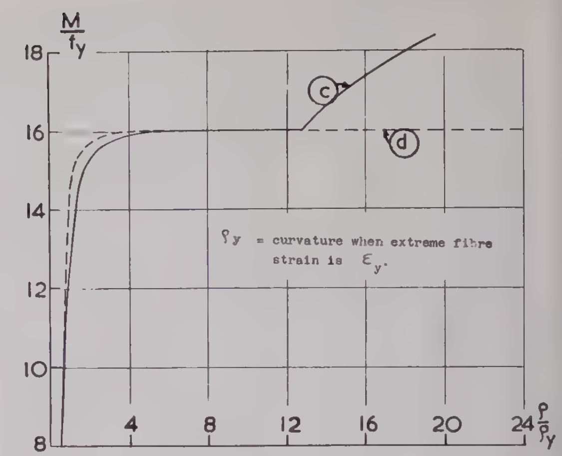

Fig. 4

3. Properties of Joist Section

A comprehensive series of flexural tests was carried out on the batch of 8 in. × 4 in. @ 18 lb. R.S.J. from which the portal frames were fabricated, and results were also available from a number of tension tests on specimens taken from various positions in the cross section of the joist. A full account of this work has been given elsewhere[2, 3]. It was found that the average properties of the steel could be represented by the stress/strain curve (*a*) in Fig. 3 where it is compared with the idealized form (curve (*b*)) usually taken as a basis for the simple plastic theory. Curve (*a*) agrees closely with the average of observed values of the stresses at the limit of proportionality ($f_p =$ 9·5 tons/in.[2]) and at yield ($f_y =$ 16·96 tons/in.[2]) and of the strain at the inception of strain hardening ($\varepsilon = 12 \cdot 7\, \varepsilon_y$). It should, however, be mentioned that in the range f_p to f_y, the strains have been slightly increased to allow to some extent for the effect of residual stresses when using the curve as a basis for determining deformations of the joist.

If now a range of bending stress distributions across the section of the joist are drawn in conformity with curve (*a*), a moment/curvature diagram can be obtained by graphical integration and expressed as curve (*c*) in Fig. 4. The corresponding relationship according to the simple plastic theory based on curve (*b*) is shown as curve (*d*) and corresponds to a full plastic moment (M_p) of 271·2 ton-in.

This information can be used to obtain values of deformations. Consider for example the cantilever CD loaded as shown in Fig. 5 where the normal bending moment diagram is CED ; let it be assumed that from F to D the moments exceed the value M_L (134·9 ton-in.) at which the limiting stress f_p is developed in the extreme fibre. In this range the moment/deformation relationship will be non-linear and it is convenient to replace the static bending moment M at each section by an equivalent moment M_E, namely the moment which would produce the same curvature in the joist had it remained fully elastic. Values of this moment can readily be determined from curve (*c*) in Fig. 4 and may be expressed as,

$$M_E = \alpha M \quad \text{(i)}$$

Values so obtained for the cantilever would be represented by the curve *ab* in Fig. 5. In this way the problem can be treated as one of elastic behaviour ; applying the simple moment-area principles to the modified bending moment diagram C*ab*D, the tip deflection is given by,

$$y = \frac{Pl^3}{3EI} + \frac{A}{EI}(l - \bar{x}) \quad \text{(1)}$$

where

$$A = \beta f_y n \quad \text{(ii)}$$

$$\bar{x} = \gamma n \quad \text{(iii)}$$

Values of α, β and γ in expressions (i), (ii) and (iii) obtained numerically from the moment/curvature curve (*c*), are plotted for the 8 in. × 4 in. @ 18 lb. joist in Fig. 6 and have been used in calculating the deformations of the portal frames.

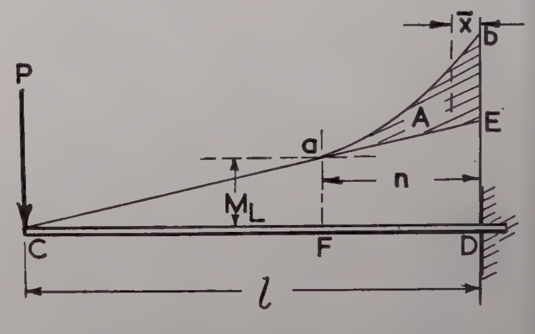

Fig. 5

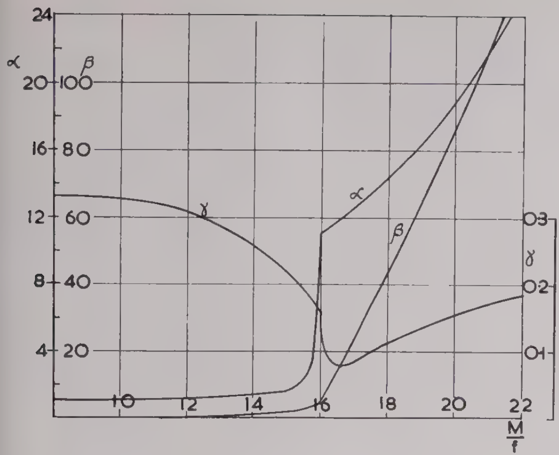

Fig. 6

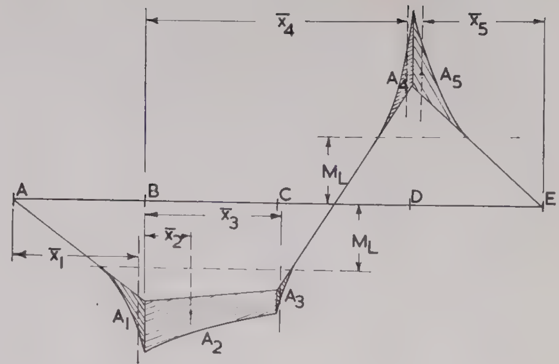

Fig. 8

4. Pinned Base Portal Frame

The deformations of a rigid frame in the partially plastic state will naturally be somewhat greater than would have been the case had the frame remained elastic, and therefore in attempting an accurate analysis account should be taken of the influence of these deformations on the distribution of bending moments. Referring to Fig. 7, the unknowns in the solution of the pinned base portal frame are the bending moments at B and D which may be determined from an equation based on the elasto-plastic deformations, and the equilibrium equation,

$$M_B - M_D = W(l + \delta) \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (2)$$

where the moments are taken to be positive when they produce tension in the extreme fibre on the inside of the frame. The term $W\delta$ represents the additional bending moment produced by the vertical reactions V_A and V_E in association with the side sway deflection and is described as a deformation moment.

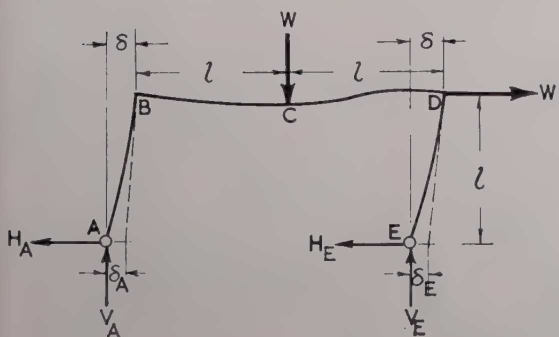

Fig. 7

The bending moment diagram for the frame, modified as described above to take account of the elasto-plastic lengths, is that shown in Fig. 8. Referring to Fig. 7, the rotations of the joints B and D can be expressed as,

$$\theta_B = \frac{1}{l}(\delta - \delta_A)$$

$$\theta_D = \frac{-1}{l}(\delta - \delta_E)$$

where δ_A and δ_E are the displacements of the tangents drawn at B and D, relative to the stanchion bases. Hence,

$$\theta_B + \theta_D = \frac{1}{l}(\delta_E - \delta_A) \quad \cdot \quad \cdot \quad \cdot \quad (iv)$$

Applying the moment-area principles to the diagram in Fig. 8,

$$EI(\theta_B + \theta_D) = \frac{Wl^2}{2} + (M_B + M_D)l + A_2 + A_3 + A_4$$

$$EI\delta_A = \left(\frac{M_B l^2}{3} + A_1\bar{x}_1\right)$$

$$EI\delta_E = -\left(\frac{M_D l^2}{3} + A_5\bar{x}_5\right)$$

and after substitution of these expressions, equation (iv) becomes,

$$\frac{Wl^2}{2} + (M_B + M_D)\frac{4l}{3} + A_1\frac{\bar{x}_1}{l} + A_2 + A_3 + A_4 + A_5\frac{\bar{x}_5}{l} = 0 \quad \cdot \quad \cdot \quad (3)$$

where the areas A have the same signs as the bending moments with which they are associated.

The sway (δ) and the vertical deflection (δ_c) under the load on the beam can be derived in the same manner and have the following values :

$$\delta = -\frac{l}{EI}\left[\frac{Wl^2}{4} + (M_B + 3M_D)\frac{l}{3} + \frac{A_2\bar{x}_2}{2l} + \frac{A_3\bar{x}_3}{2l} + \frac{A_4\bar{x}_4}{2l} + \frac{A_5\bar{x}_5}{l}\right] \quad \cdot \quad (4)$$

$$\delta_c = \frac{l}{EI}\left[\frac{Wl^2}{6} + \frac{l}{4}(M_B + M_D) + \frac{A_2\bar{x}_2}{2l} + \frac{A_3(2l - \bar{x}_3)}{2l} + \frac{A_4(2l - \bar{x}_4)}{2l}\right] \quad (5)$$

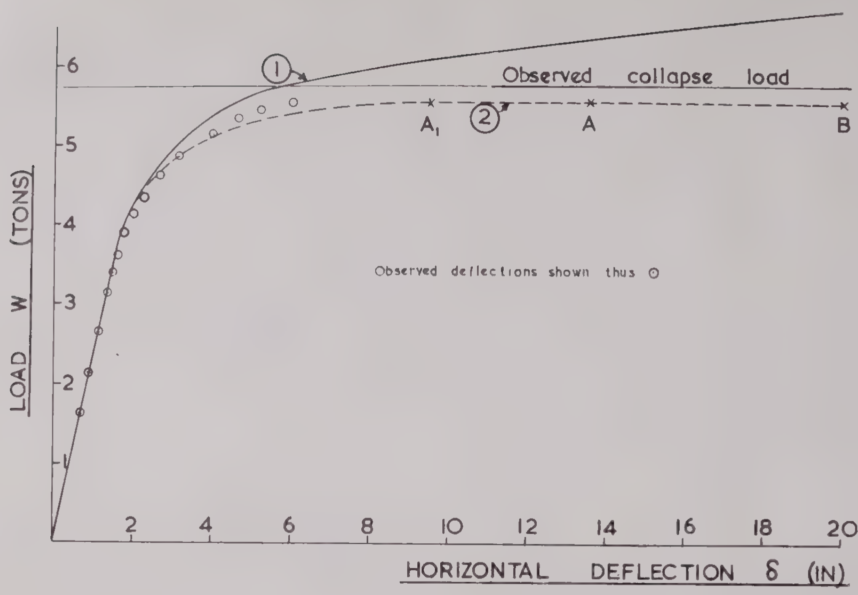

Fig. 9

In the first examination of the frame no account was taken of the side sway deflection in determining the moment distribution. The procedure was simply to assume a value of M_B for a given value of W, then determine the corresponding value of M_D from equation (2) with $\delta = 0$, and finally to ascertain whether these moments would satisfy equation (3) in which the terms A and $\bar{x}$ were evaluated with the aid of the curves in Fig. 6. Usually acceptable values of the moments could be determined after two or three attempts. Corresponding values of δ and δ_c were then obtained from equations (4) and (5).

These values of δ, shown in Fig. 9 as curve (1), are somewhat less than those observed during the test on the portal for which catastrophic collapse occurred at $W = 5 \cdot 75$ tons, as compared with a value of $5 \cdot 65$ tons calculated according to the simple plastic theory[1]. It will be seen that the theoretical curve shows no sign of failure at these loads. In fact at the last point calculated ($W = 6 \cdot 75$ tons, $\delta = 7 \cdot 04$ in.), there was every indication that the rate of increase in moment of resistance at the partially plastic sections due to strain hardening, should have been sufficient to enable the frame to support considerably more load.

It was therefore evident that the influence of deformation on the moment distribution must have been a significant factor in determining the nature of failure of the frame. Due account has been taken of this in the second analysis. For a given value of W, values of both M_B and δ were assumed in determining M_D from equation (2). For each value of δ, values of M_B and M_D satisfying equation (3) were obtained by trial and error. These were then used to determine a more accurate value of δ from equation (4). By a process of iteration it was thus possible in a few cycles to arrive at acceptable values of both the moments and the side sway deflection for loads up to $W = 5 \cdot 55$ tons corresponding to $\delta = 9 \cdot 46$ in. and shown as point A_1 on curve (2) in Fig. 9. However,

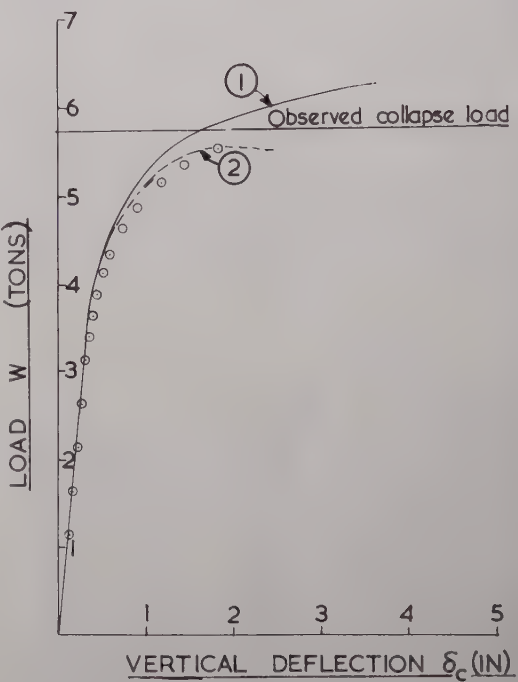

Fig. 10

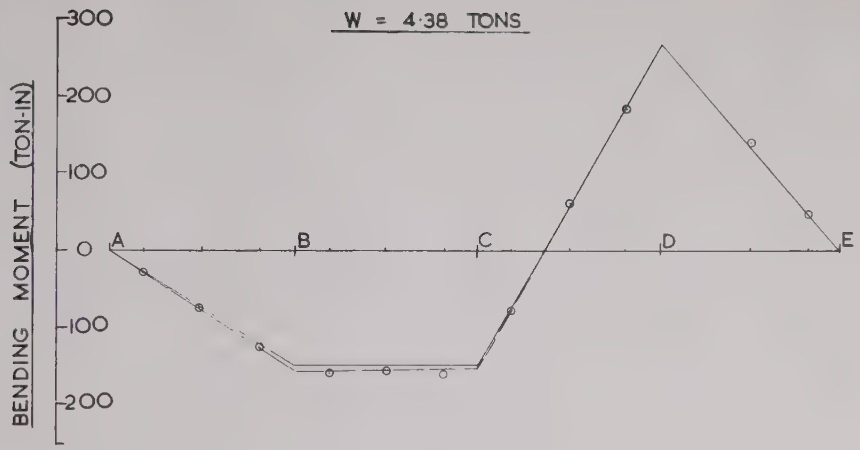

Fig. 11

at $W = 5{\cdot}75$ tons, the observed collapse load, it was found that as the cycles of the calculation were repeated, the closing error in δ began to diverge rapidly with consequent large increases in its actual value. Evidently the frame was theoretically incapable of sustaining this load. Conditions of equilibrium could, however, be established at values of δ in excess of $9{\cdot}46$ in. by reducing the load on the frame as represented by the portion AB of the curve (2) in Fig. 9 where the points A and B represent respectively the values $W = 5{\cdot}55$ tons, $\delta = 13{\cdot}5$ in. and $W = 5{\cdot}52$ tons, $\delta = 20{\cdot}0$ in. It should be emphasized that towards the end of the test increments of $0{\cdot}2$ tons of dead load were being applied; the frame supported loads of $5{\cdot}55$ tons but collapsed suddenly when the next increment had been added. There would seem to be little doubt that the catastrophic failure was brought about by a condition of general instability as indicated by the crest in the theoretical curve. A similar explanation of behaviour could have been given from a study of the vertical deflections (δ_c) in Fig. 10 where observed values are compared with curves (1) and (2) representing values calculated by first neglecting, and then taking into account the side sway, respectively.

It is also of interest to examine the distribution of bending moment in the frame when partially plastic. In the test[1] four Maihak gauges were fitted at each of a number of sections away from the plastic zones so that bending moments could be deduced well into the loading range after yielding had commenced. In converting the observed strains into stresses in the non-linear range (i.e. for stresses in excess of f_p) values were taken direct from the curve in Fig. 3. At a loading of $W = 4{\cdot}38$ tons all but one of the sets of gauges were operating; corresponding values of bending moments are shown as circles in Fig. 11 and are compared with theoretical values shown by the full line (neglecting sway) and by the broken line (taking account of sway). The effect of sway is small at this stage of the loading but even so the latter solution is obviously the more accurate.

At a load immediately prior to the last increment, namely $W = 5{\cdot}55$ tons, 5 sets of gauges were still operating; the corresponding bending moments are compared with theoretical values in Fig. 12. The effect of sway is naturally greater in this case and again there is evidence of better agreement with the analysis in which it is taken into account.

5. Fixed Base Portal Frame

The method of analysis used in examining the pinned base portal could, of course, be extended to cover the case of the fixed base frame, but the necessity of varying four moments in the trial and error procedure makes it extremely tedious. A more manageable method is a version of the slope-deflection approach devised by Rawlings[4] in which the relationships between terminal moments and end slopes of the partially plastic lengths are first determined, and then by trial and error a bending moment distribution for the frame is found for which end slopes and deflections are compatible.

This task is made easier in the present case by the fact that the partially plastic lengths between moment peaks AB, BC, CD and DE are all of the same length, namely 96 in. The relationships between terminal moments and slopes for this length when bent in double curvature as at (*a*) in Fig. 13, may be obtained from a consideration of the modified bending moment diagram (*b*). The slopes can be written as,

$$\theta_{AB} = \frac{1}{EI}\left[\frac{l}{6}(2M_A + M_B) + \frac{A_1(l - \bar{x}_1)}{l} + \frac{A_2\bar{x}_2}{l}\right] \quad . \quad (6)$$

$$\theta_{BA} = -\frac{1}{EI}\left[\frac{l}{6}(M_A + 2M_B) + \frac{A_1\bar{x}_1}{l} + \frac{A_2(l - \bar{x}_2)}{l}\right] \quad . \quad (7)$$

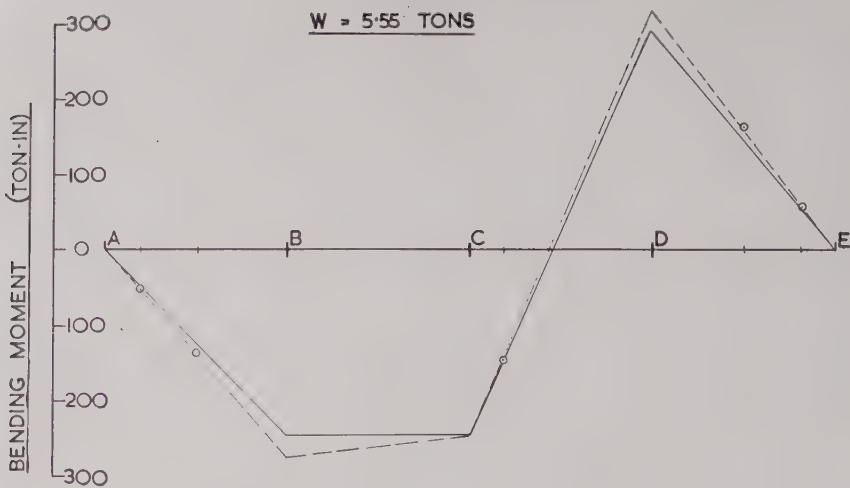

Fig. 12

where hogging bending moments are taken to be negative and the terms A have the same sign as their corresponding moment. From these and similar expressions for the case of single curvature bending, a family of curves, of which those in Fig. 14 are typical, were obtained relating terminal moments to end slopes for the 96 in. length of 8 in. × 4 in. joist.

For the fixed base frame, the horizontal equilibrium equation including the effect of the side sway deflection can be expressed as,

$$M_A - M_B + M_D - M_E = -W(2l + \delta) \quad \cdot \quad (8)$$

In obtaining the additional equations necessary for the evaluation of the moments, it is convenient to write the numerical values of the rotations at A, B . . . as θ_A, θ_B . . . and those of the corresponding slopes of members relative to the line joining the extremities of each of the lengths AB, BC . . . as θ_{AB}, θ_{BA}; θ_{BC}, θ_{CB}; . . . It will be evident from Fig. 15 that,

$$\left.\begin{aligned} l(\theta_A + \theta_{AB}) &= \delta \\ l(\theta_E + \theta_{ED}) &= \delta \end{aligned}\right\} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (9)$$

or

$$(\theta_A + \theta_{AB}) = (\theta_E + \theta_{ED}) \quad \cdot \quad \cdot \quad \cdot \quad (10)$$

Also,

$$\left.\begin{aligned} l(\theta_B - \theta_{BC}) &= \delta_c \\ l(\theta_{DC} - \theta_D) &= \delta_c \end{aligned}\right\} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (11)$$

so that,

$$(\theta_B - \theta_{BC}) = (\theta_{DC} - \theta_D) \quad \cdot \quad \cdot \quad \cdot \quad (12)$$

in which

$$\theta_B = \theta_A + \theta_{AB} - \theta_{BA}$$

and $$\theta_D = \theta_E + \theta_{ED} - \theta_{DE}.$$

It will be seen that in the above equations account has been taken of possible rotation at the stanchion bases. This is necessary in the present case since the compound girder to which the stanchion bases were attached (Fig. 2) did, in fact, allow significant rotation to occur. A mirror was fixed to the centre of the web at the base of the windward stanchion and the rotations were recorded with a telescope and fixed scale. The bending moments transmitted through this connection between stanchion and compound girder were available from the readings of the Maihak gauges so that a moment/rotation curve could be plotted as shown in Fig. 16. This relationship has been assumed to apply to both stanchion bases.

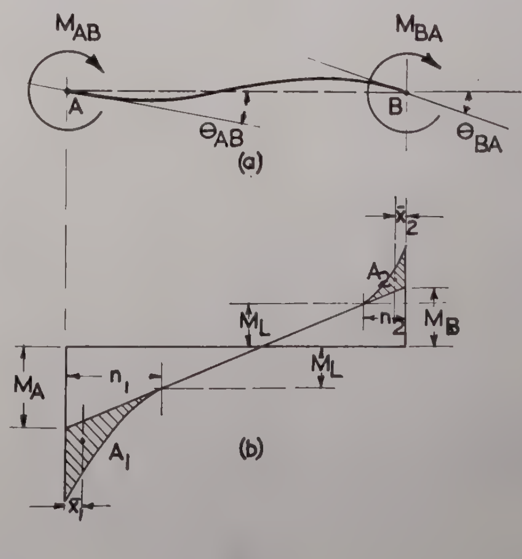

Fig. 13

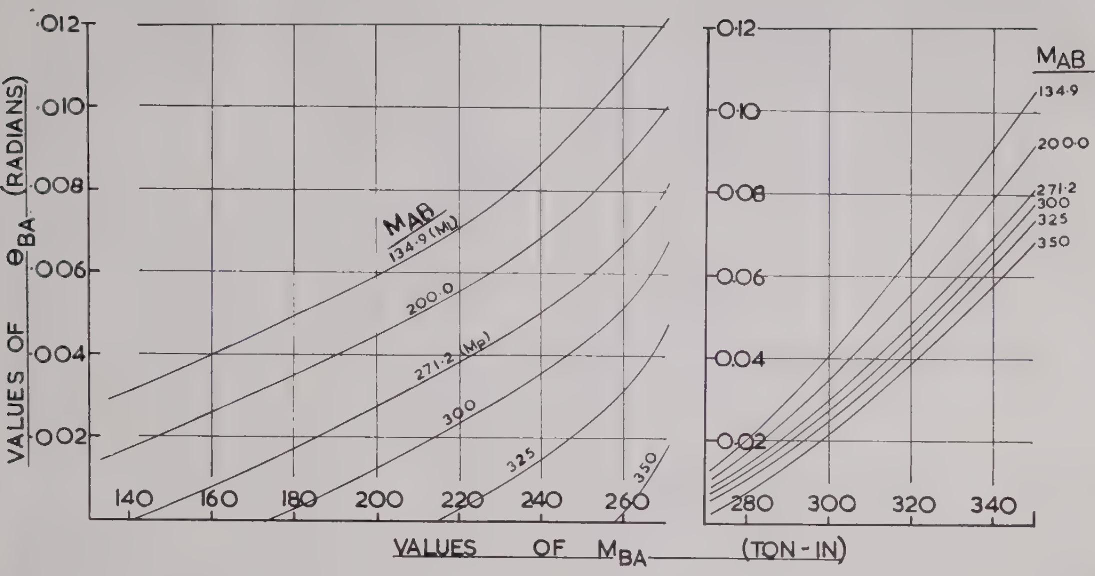

Fig. 14(a)

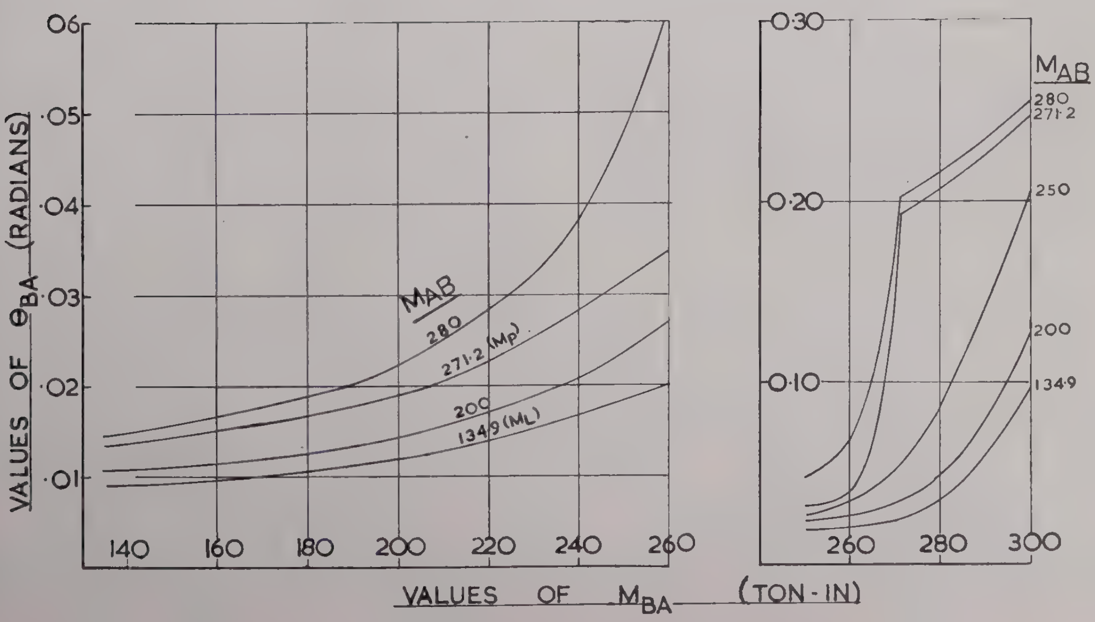

Fig. 14(b)

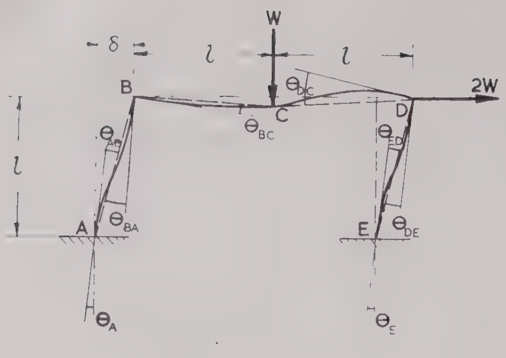

Fig. 15

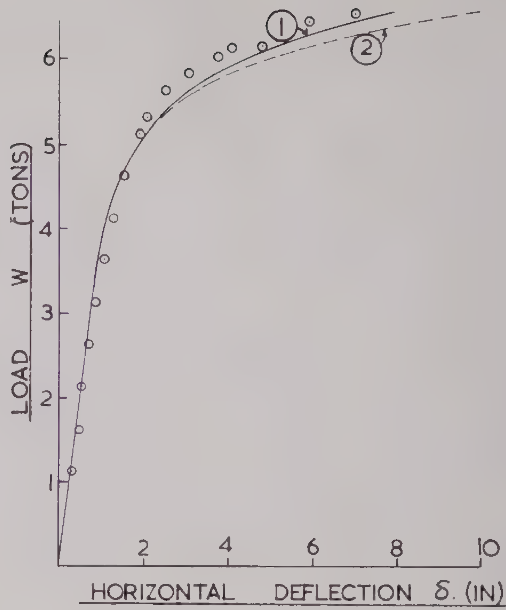

Fig. 17

Again, in the first examination of the frame no account was taken of the side sway deflection in determining the moment distribution. The procedure was to assume a value of $(M_A - M_B) = M$ for a given value of W and then to determine a corresponding value of $(M_D - M_E) = N$ from equation (8) with $\delta = 0$. Using the curves in Fig. 14 and relating M_A to θ_A by means of the curve in Fig. 16, the component moments of M were chosen to give a reasonable value of δ as expressed by the first of equations (9). The components of N could then be varied until equation (10) had been satisfied. Finally the values of M_A, M_B, M_D and M_E so determined were checked by means of equation (12), again with the aid of the curves in Fig. 14. If this equation were not satisfied, a closer value of M was selected and the process repeated until a correct solution was obtained. Once the moments were known, values of δ and δ_c could be deduced from equations (9) and (11) respectively.

Values of the side sway deflection δ so obtained are plotted as curve (1) in Fig. 17 and are compared with experimental values shown as circles. The latter, though somewhat less than the former, show the same general trend as the theoretical values. It was estimated according to the simple plastic theory that collapse should have occurred at $W = 5{\cdot}65$ tons, but as the experimental points show the frame continued to carry loads above this value. At $W = 6{\cdot}13$ tons the test was discontinued for the day. A reading taken on the following morning showed that the horizontal deflection had increased overnight by about 18 per cent. Additional loading was then applied but had to be discontinued at $W = 6{\cdot}55$ tons, the limit of loading capacity. This load was left on for three days but showed no signs of causing collapse. This is borne out by curve (1) which continues to climb beyond the load of 6·55 tons corresponding to a value of δ of 7·71 in. as compared with an observed value of 7·02 in.

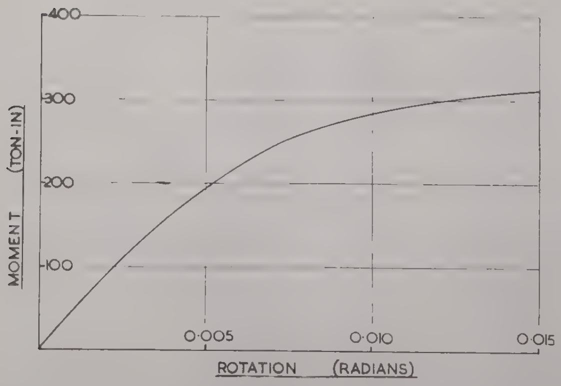

Fig. 16

However, in deriving curve (1) no account has been taken of the influence of the deflection δ on the stability of the frame. In the second treatment of the problem, the values of δ given by curve (1) were used in equation (8). The subsequent procedure was as previously described, the final step being the determination of a more correct value of δ as given by equation (9). A sufficiently accurate solution could usually be obtained after a few cycles of the process. Values of δ so determined have been plotted in Fig. 17 as curve (2) for values of W from 5·33 to 6·55 tons. Unlike the corresponding curve for the pinned base frame, this one shows no signs of a crest thus indicating that the loading condition for instability does not occur in the region of the collapse load of 5·65 tons calculated according to the simple plastic theory.

Corresponding theoretical and experimental values of δ_C are shown in Fig. 18 and lead to similar conclusions.

The highest load at which bending moments were recorded was $W = 5{\cdot}33$ tons. The observed values are shown as circles in Fig. 19; the full line represents the theoretical values where side sway deflection is neglected, and the broken line those for the case when it is taken into account. The effect of this deflection

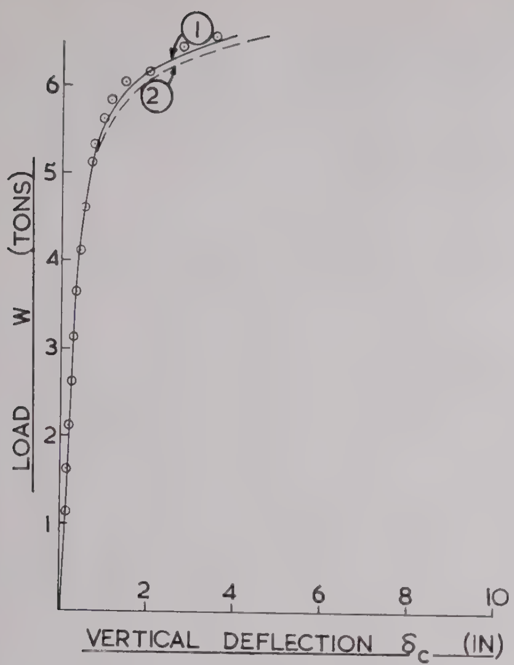

Fig. 18

on the moment distribution is not particularly significant at this load; the general agreement between theoretical and observed values is good with the possible exception of those values in the length BC. It should however be mentioned that the vertical deflection (δ_c) has some influence on the moment distribution over this length. Considering the vertical equilibrium of the frame, it can be shown that the bending moment at C is given by the equation,

$$M_C = \frac{Wl}{2} + \frac{1}{2}(M_B + M_D) + \frac{\delta_c}{l}(M_A - M_B) \qquad (13)$$

In determining the theoretical values of moments shown in Fig. 19, the term involving δ_c has been neglected. As may be seen from the following Table the effect of including this term is to reduce M_C by about 2 per cent without causing any significant change in the moments at top and bottom of stanchions.

Bending Moments (ton-in.) for $W = 5 \cdot 33$ tons

δ (in.)	δ_c (in.)	M_A	M_B	M_C	M_D	M_E
Neglected	Neglected	276	186	208	281	280
2·46	Neglected	278	195	212	282	282
2·46	0·86	278	195	208	282	282

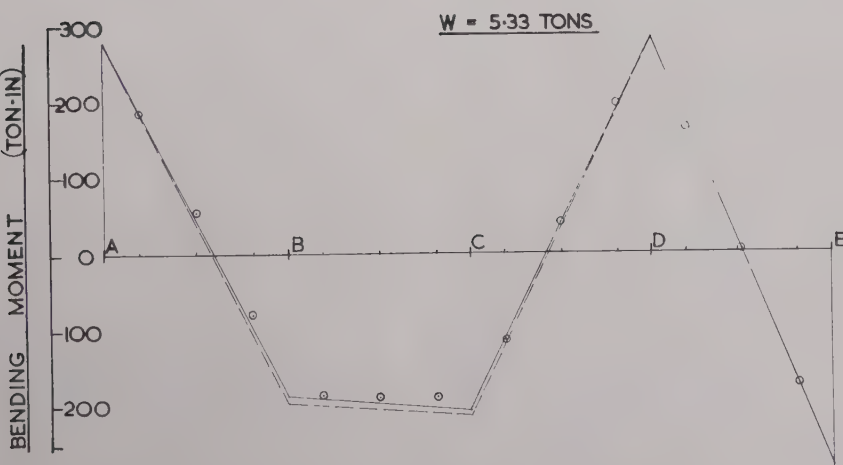

Fig. 19

6. Conclusions

(1) The accuracy of the method used for determining deflections is surprisingly good considering that it is based on a simple stress/strain relationship representing the average properties of the material over the whole cross section of the joist, and involves an integration over the length of each plastic zone. In its more accurate form where account is taken of the effect of the side sway deflection on the moment distribution, the theoretical values obtained are somewhat greater that those observed at the higher loads. In this connection, however, it must be remembered that the intention was to keep the duration of the tests within the limits of daylight and it was therefore not possible to exhaust the whole of the creep which could occur after the application of each increment of load. This effect would tend to be accentuated as the number of plastic zones increased and this would account for the greater difference between theoretical and observed values in the case of the fixed base frame.

(2) By far the most useful outcome of the work is that of establishing more closely the character of the load/deflection relationship for a rigid frame structure in the vicinity of the collapse load as determined by the simple plastic theory. It becomes clear from the analysis that only the simplest of these structures will exhibit the characteristic of a simple beam in pure bending whereby a maximum loading condition is reached at which the member deflects continuously at a more or less constant rate. In general where the strength of the structure depends only upon its flexural properties, the behaviour will be typical of the fixed base frame, where strain hardening enabled the plastic sections to develop moments greater than the so-called 'full plastic value' and permitted the frame to carry loads in excess of the calculated collapse load. On the other hand, a structure may be sufficiently flexible for the deformation moments to increase at a greater rate than that of the growth of the moments of resistance due to strain hardening at the positions of peak moments. In this case the load/deflection curve will eventually show the drooping characteristic as in Fig. 9 and sudden failure will occur at the load determined by the crest of the curve. In the case of the pinned base frame the calculated collapse load and this load for instability were not very different ; they were, in fact, 5·65 tons and a critical load slightly in excess of 5·55 tons. However, as later experience has shown, the critical load can at times be significantly lower than the calculated collapse load, particularly for frames with tapered sections now being studied at the University of Sydney[5].

(3) Another point worthy of mention is the actual moment distribution, and in particular the values of the individual peak moments, at or near the predicted collapse load. According to the simple plastic theory both frames should have collapsed under loadings corresponding to $W = 5{\cdot}65$ tons with individual peak moments limited to the full plastic value of 271 ton-in. The greatest loading supported by the pinned base frame was $W = 5{\cdot}55$ tons when the peak moments as determined by the more accurate analysis, are those given in the accompanying Table (see also Fig. 12). Evidently the combined effect of the deformation moments and strain hardening was to increase the moment at D by 15·5 per cent above the full plastic value of 271·2 ton-in.

Portal Frame	Load W (tons)	δ (in.)	Theoretical Values of Bending Moments (ton-in.)				
			M_A	M_B	M_C	M_D	M_E
Pinned Base	5·55	9·46	0	273	246	313	0
Fixed Base	5·65	3·36	289	230	241	291	293

A similar effect may be noted in the corresponding values of moments given in the Table for the fixed base frame, the maximum increase over the full plastic value being approximately 8 per cent at E. The lower value, in this case as compared with the pinned base frame, results no doubt from the reduction in deformation moments.

The possibility of the peak moments exceeding the full plastic value in this way at the predicted collapse load is of importance in the design of welded splices and connections. Obviously where these coincide with sections likely to become fully plastic, all welds should be proportioned so that moments well in excess of the full plastic value can be transmitted without any danger of fracture.

The author wishes to express his thanks to Mr. D. F. Rogers, B.E., Technical Officer in the Department of Civil Engineering, University of Sydney, for his assistance with the computations for the fixed base portal frame.

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The Council will be glad to consider the publication of correspondence in connection with the above paper.
Communications on this subject intended for publication should be forwarded to reach the Institution by the 31st October, 1960.

Some Points of Structural Interest at Calder Hall 'A' Nuclear Power Station

by

W. S. Watts, M.I.Struct.E., A.M.I.C.E.

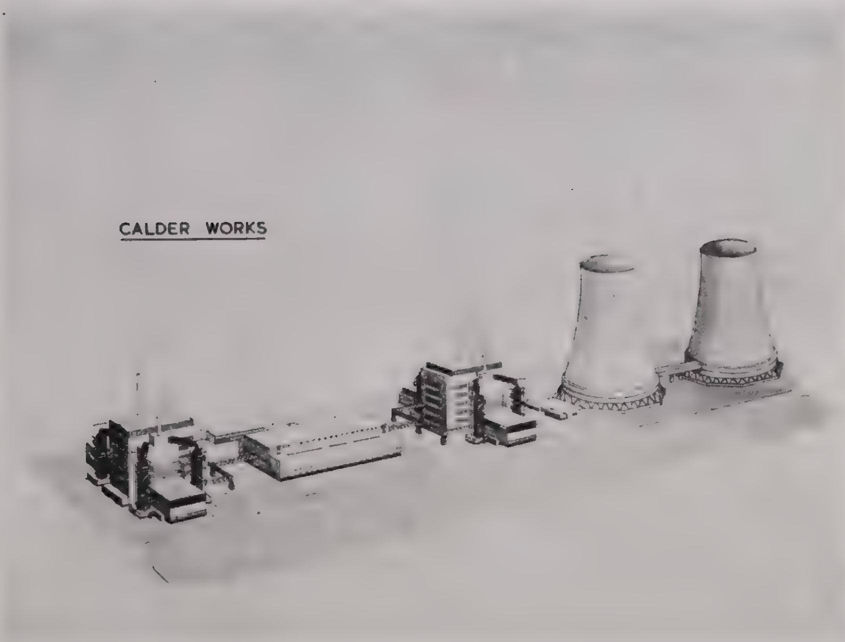

Fig. 1.—General arrangement of Calder Hall 'A' Nuclear Power Station.

Synopsis

The development of atomic reactors as an alternative source of power for use in electricity generating stations led to the design and construction, by the United Kingdom Atomic Energy Authority, of Calder Hall 'A' Nuclear Power Station which is situated on the opposite bank of the River Calder to the Authority's Windscale Works in Cumberland.

The project included two reactors requiring large foundations, concrete biological shield walls and associated steel framed buildings. These, together with a turbine hall, administration building, service ducts, bridges and two cooling towers with their services, comprise the complete station (Fig. 1).

No explanation is given of atomic physics or of the intricate points of reactor design, but the principal features of the project are outlined, together with an account of some of the points of structural interest, including illustrations showing the work in progress on site.

Introduction

The design of the project, and particularly the part involving the reactors, was liable to have an appreciable amount of day to day modification upon it, resulting in a very tight programme for the detailing and construction. The Civil Engineering Contract was let on a prime cost basis in order that construction could immediately follow the final design, and to permit the progressive issue of detailed working drawings to the site.

The early months of 1953 saw the start of construction with a road bridge over the River Calder giving access to the new site. By midsummer of that year the contract was let for the civil and structural engineering work, calling for an immediate start. The master programme showed that this work had to be completed in a period which would allow the station to be opened officially before the end of 1956.

The value of the Civil Engineering Contract for the project was approximately £2·5*M* and a further appreciation of the work involved may be gained from the fact that there were some 150,000 cu. yds. of excavation, 3,300 tons of structural steelwork and 75,200 cu. yds. of concrete using 2,500 tons of reinforcement. At the height of the contract the civil labour force consisted of approximately 750 men including, at one stage, 100 scaffolders.

A breakdown of these figures shows that each of the two reactors required approximately 7,000 cu. yds. of excavation and 18,000 cu. yds. of concrete together

Fig. 2.—The Road Bridge over the River Calder.

with 700 tons of reinforcement. The steel framed building associated with the reactor took a total of 900 tons of structural steelwork. The turbine hall included 800 tons of structural steelwork, with considerable quantities of reinforced concrete in the foundations and ductwork; each set of turbo-blocks required 300 cu. yds. of reinforced concrete. The two cooling towers are approximately 300 ft. high with a base diameter of 200 ft. and each required 4,000 cu yds. of concrete and 400 tons of reinforcement.

Having briefly surveyed the extent of the work, particular aspects of the more important features will be described.

The Road Bridge over the River Calder

Unlike the more conventional electricity generating station, rail sidings are not necessary to the Calder Works and a road bridge (Fig. 2) forms the only access from the existing country roads, and from the Windscale Factory, to the new site. This bridge was a necessary preliminary to the main construction and had to be capable of taking all types of loading during the construction period, and subsequently when the station is in operation. It was, therefore, designed for the M.O.T. standard loading.

The bridge has two outer spans each of 28 ft., with a central span of 34 ft.; the deck, 16 ft. above the level of the stream bed, carries two lanes of traffic and has a cantilevered footpath at each side of the roadway. The concrete abutments, faced with local stone to blend with the natural surroundings, and the intermediate reinforced concrete piers sited in the river, were founded on sandstone and construction was carried out using a 1 : 2 : 4 mix with rapid hardening cement.

A composite construction using 24 in. $\times$ 7½ in. R.S.J.'s with an 11 in. thick concrete slab was used for the bridge deck. Reinforcing bars were secured to the top flange of the joists, at pitches dictated by the design, to take care of the shear between the concrete slab and the steel beams and so ensure the desired composite action. Construction was simplified by using permanent pre-cast concrete shutters across the top flanges of the deck beams, an arrangement which also provided facilities for carrying service pipes and cables in a concealed position under the deck between the webs of the joists. The deck and footpaths were constructed in a 1 : 2 : 4 nominal mix with Portland cement, having a specified cube strength of 3000 lb./sq. ins. at 28 days.

In order to give a more pleasing appearance, and to provide a measure of protection, the underside of the cantilevered footpaths were haunched down along each side of the bridge to form a casing to the outer deck joists.

A 3 in. wearing surface of tarmacadam, laid in two courses, was provided for the roadway, whilst the concrete surface of the footpaths was patterned by a pin-head roller.

The Reactors and Ancillary Buildings

A simple diagram of a nuclear power station (Fig. 3) shows a basic similarity to coal or oil fired stations. At Calder Hall the heat is produced by the fission of the uranium fuel elements within the reactor core, this heat is gathered by a coolant, in this case carbon-dioxide (CO_2), which is pumped through the reactor and then into the heat exchangers, or boilers, via a closed circuit.

The heat exchanger is basically a means of transferring the heat from the coolant to the water, thus raising steam in the H.P. and L.P. circuits. The steam is then passed to the turbines, so providing the necessary power for generating electricity.

The steam load of the heat exchanger, which is in circuit with the reactor, can be kept constant regardless of the load on the turbo-alternator and to permit this a " dump-condenser " is provided in the steam circuit. The dump-condenser is a large diameter drum, honeycombed with tubes through which water passes, and steam passed into the drum is condensed by the cold tubes. In this way it is possible to deal with all or any part of the steam raised in the heat exchangers which would otherwise pass to the turbines.

The water used in the cooling circuit is pumped through the cooling towers to achieve the necessary temperature drop before re-circulating.

Structural Aspects of the Reactors

At Calder Hall the design of the vessel for containing the reactor core was not a structural responsibility, and attention will be focused on the raft foundation, biological shield and roof, the control, reactor cap and discharge buildings together with the blower houses (Figs. 4 and 5).

The main structural problems associated with the Calder Hall reactors were :—

(i) the provision of suitable shielding against harmful radiations and

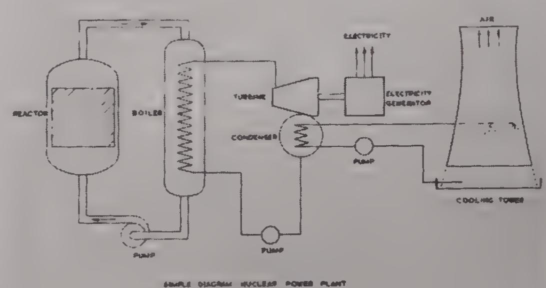

Fig. 3.—Flow Diagram for Nuclear Power Station.

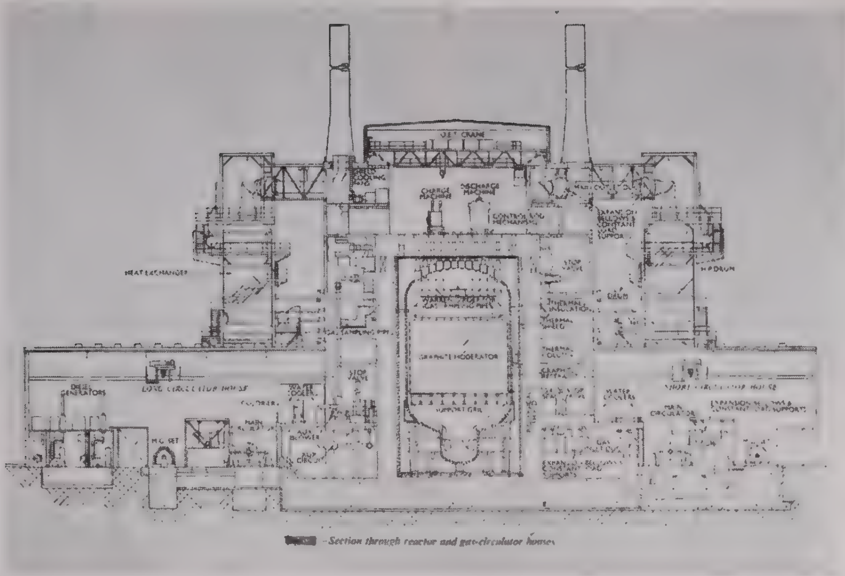

Fig. 4.—Longitudinal Section through Reactor showing blower houses.

(ii) the design of the foundations for the large concentration of loading in the area surrounding the reactor vessel.

The reactor core is gas cooled and graphite moderated and contained in a 2 in. thick welded steel plate pressure vessel, which is housed within the concrete biological shield. The dead weight of the vessel, together with the loads from the graphite core, fuel elements etc., is transmitted to the main raft foundation through grillages.

Harmful radiations are emitted from the reactor core following the fission of the uranium fuel and hence some form of shielding is required. To overcome the problems at Calder Hall an inner thermal shield of 6 in. thick steel plate and an outer reinforced concrete biological shield were provided. The thickness of the concrete for shielding purposes had to be 7 ft. for the walls and 8 ft. for the roof.

Ideally the biological shield would be a complete box surrounding the thermal shield, but in practice the box must be pierced by a large number of holes for ducts, charge and discharge mechanisms, inspection tubes, etc. In addition to its shielding purpose the concrete biological shield must carry both its own weight and all other applied loads, and effectively transmit them to the foundation raft.

Normal reinforced concrete is the cheapest material which almost meets the ideal and economical requirements for the biological shield of a reactor producing power on a commercial scale, and the shield at Calder Hall was designed on this basis. It is also an advantage to be able to use the outer faces of the biological shield to form one side of the adjacent control, discharge buildings and blower houses (Figs. 4 and 5), and again at Calder Hall, conditions were such that this course could be adopted without adverse effect in any respect.

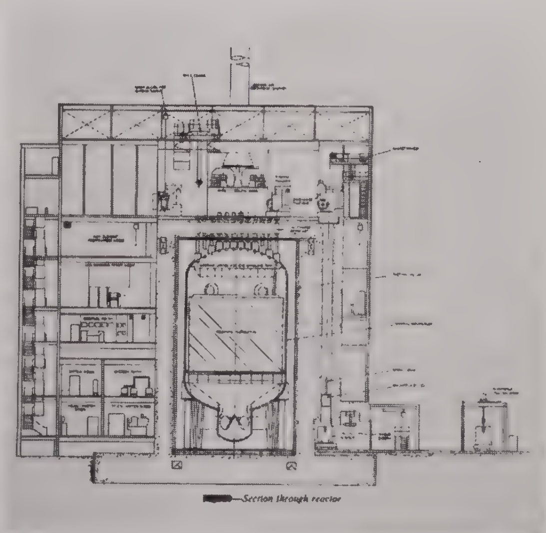

Fig. 5.—Cross Section through the Reactor showing Control and Discharge Buildings.

As previously stated, the main factor influencing the design of the foundations was the large concentration of load within the area of the biological shield; approximately 20,000 tons being concentrated over an area which would have given a pressure in the region of 6 tons/sq. ft. Site investigations indicated that the raft would be founded in a glacial moraine of sand and gravel with clay lenses. The total depth of this

Fig. 6.—View of the Raft Foundation during Construction.

strata was about 60 ft. and below this level was a bed of sandstone rock. Bearing in mind the desirability of restricting differential settlements to a low value, the permissible ground pressure at formation level, i.e., 15 ft. below normal ground level, was assessed at 2·5 tons/sq. ft. Using this bearing pressure the area of the foundations was determined, and resulted in a raft 11 ft. thick, with a total load of 34,000 tons being transmitted to the underlying strata over an area of 13,500 sq. ft.

When the foundation raft was constructed four brass levelling pegs were set in special housings, and settlement readings were taken continually throughout the construction period as the loading progressively increased. The maximum settlement, recorded over the construction period of approximately $2\frac{1}{2}$ years, was less than $\frac{5}{16}$ in. and was uniform over the area of the raft.

Construction of the Reactors

The top soil was removed by the use of tractors and scrapers, and placed on soil dumps for future use.

The main excavation of approximately 7,000 cu. yds. to each reactor, was carried out using a $1\frac{1}{4}$ cu. yd. dragline, the sides of the excavation being battered to a slope of 1 in $1\frac{1}{2}$. Immediately the excavation was completed over a particular area, oversite concrete was laid to protect the exposed surface. This procedure was repeated until the whole of the excavation was completed and the entire surface protected.

Permanent brick shuttering (Fig. 6) was built on the oversite concrete to form the perimeter of the raft which was subsequently constructed in four lifts, each lift being poured in alternate panels or bays with the use of precast concrete stop-ends. The upper lift of the raft was provided with a 6 in. deep upstand, or ' kicking piece,' to receive the bottom of the shutters for the first lift of the biological shield. Altogether a total of 264 tons of reinforcement and 5,200 cu. yds. of concrete were used in each raft. The time taken to construct a raft was about six weeks and at the peak of the work a pouring rate of 140 cu. yds. of concrete per day was achieved.

It was decided from the practical point of view that the most advantageous shape for the main biological shield was an octagon. In addition, further shield walls occur forming ' wings,' or an ' envelope,' on two sides of the main biological shield. As already mentioned, a load of 20,000 tons is concentrated over the area of the octagon. In order to spread this load over a rectangular raft developing a foundation area of 130 ft. × 104 ft., while still keeping the thickness in reasonable proportions, the wing or envelope walls were used as large stiffening ribs or gussets. These walls were, therefore, constructed as an integral part of the main octagon.

The 85 ft. high biological shield walls were divided in plan into a total of 18 bays for the purpose of concreting (Fig. 7). There were eight bays to the octagon and five bays to each of the subsidiary shield walls or envelopes.

Floors, occurring at four levels within the envelope walls (Fig. 4), were omitted during the main wall construction so as to speed up the programme for the octagon. These floors were subsequently carried on R.S.J.'s supported by brackets, which were in turn welded to steel wall plates cast in during the construction of the main walls. The envelope walls rise higher than the main biological shield walls and the concrete roof over provides seating for the 90 ft. high self-supporting steel ventilation shafts for the air cooling circuit.

In the height of the octagon 20 lifts of 4 ft. 6 in. each were made. The upper lifts were stepped back in thickness from the inside face to provide seating for the various lifts of the biological shield roof.

A high degree of accuracy on the inner face of the octagon was specified in order that the 6 in thick steel lining would fit successfully, so maintaining the necessary air passage for cooling between the steel lining and the biological shield. It also ensured that the plant items which were to be cast within the concrete walls would be within the permitted limits. An inner face tolerance of $-\frac{1}{4}$ in. and $+\frac{3}{4}$ in. was specified and it is interesting to note that the work as constructed was well within these limits. To achieve this accuracy both vertically and in plan a prefabricated steel angle shutter tower, or ' spider,' was used for

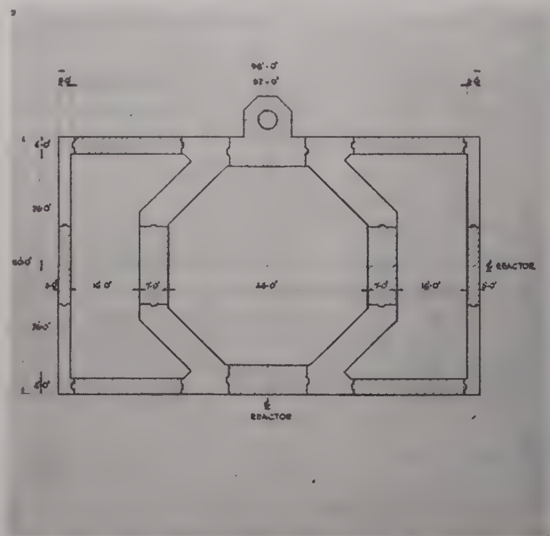

Fig. 7.—Plan to Octagon and Envelope Walls.

Fig. 8.—Internal " Spider " and shuttering to Octagon Walls.

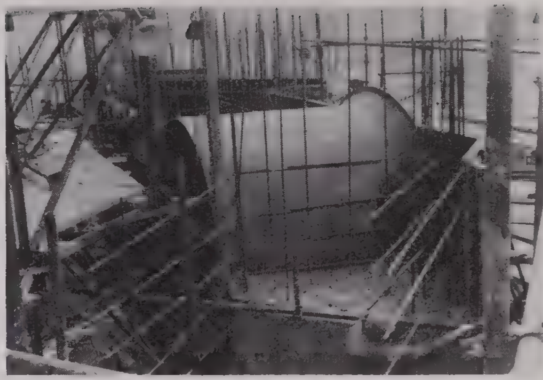

Fig. 9.—CO_2 Duct through the Biological Shield.

the full height of the walls (Fig. 8). The spider had accurately positioned steel corner and intermediate soldiers, which, as well as forming a shuttering face themselves, also acted as jigs for the infilling wooden panel shutters.

The outer faces of the walls were shuttered using 2 in. thick timber panels on steel braced arms, nut setters being used to cast nuts into each lift so allowing the shutters for the following lift to be secured. The shutters were lifted by the use of 5 cwt. chain blocks suspended from a higher point on the working scaffold adjacent to the wall. The outer faces, adjacent to the steel framed Control, Discharge and Blower House buildings, had heavy steel wall plates, complete with anchor lugs, cast in flush with the wall face. Cleats were later welded to the plates forming seatings for the main floor beams and the scantlings of the adjoining buildings. This system had advantages from the shuttering point of view and helped to speed up the work on the main biological shield. Within the octagon anchor plates were cast in at each corner, and at intermediate positions, for the attachment of solid steel soldiers to receive the 6 in. thick thermal shield plates (Fig. 12).

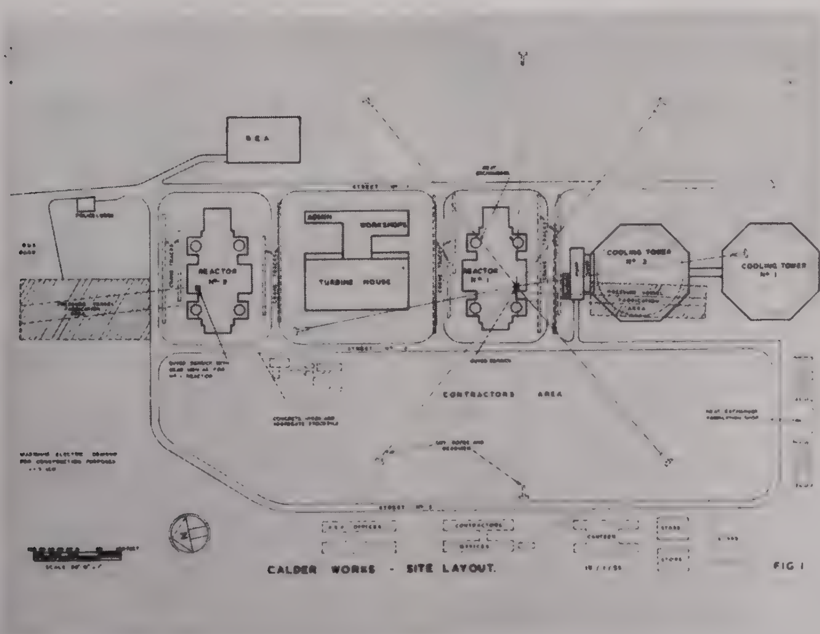

Fig. 10.—Site Layout.

Fig. 11.—The 100 ton Lifting Stick.

Incorporated in the intermediate lifts of the main shield walls were cages for the burst slug detection gear, large holes for the CO_2 pipes (Fig. 9) and numerous smaller openings, all of which presented problems relative to construction and concreting procedure.

An important characteristic of the biological shield is that it shall have as high a density as possible. The specification for the concrete required a minimum dry density of 150 lb. per cu. ft. with a 28 day cube strength of 3,000 lb. per sq. in. The only aggregate available within reasonable distance of the site which enabled these requirements to be guaranteed was Northumberland Whinstone.

In order to ensure the necessary high standard for the concrete and workmanship in the biological shield special steps were taken, and the design and control of concrete mixes was carried out in a Site Concrete Laboratory with a full time engineer in charge. Full size 'mock-ups' of certain portions of the walls were concreted, to ensure that complete compaction had been achieved and that no voids were likely to arise around the many tubes etc., cast into the walls.

A mix was designed using a low water/cement ratio to minimise shrinkage, and to assist in achieving the high density. This resulted in a mix using a 6 : 3 : 1 aggregate/cement ratio with a water/cement content of 0·52. All concrete was mechanically consolidated by the use of internal vibrators.

At the vertical ends of all wall bays male and female type construction joints were used. The horizontal joint surfaces of all lifts were thoroughly hacked by hand for a depth of at least one inch, to ensure that no straight paths would exist for any hazardous leakage.

The majority of the concrete for the whole of the contract was mixed in a central batching plant using 2—Stothert and Pitt 1 cu. yd. mixers, with separate weigh batchers for cement, sand, small and large aggregates, and with the use of calibrated water delivery gauges; the plant had storage capacity for 500 tons of cement. The concrete was transported by specially adapted tipping vehicles, which conveyed and tipped 2 cu. yds. of concrete directly into skips for hoisting and placing by derrick cranes. Using these methods the concreting reached a peak of over 2,000 cu. yds. per week with an average output of about 800 cu. yds. per week. The octagon and envelope walls took approximately seven months to complete and necessitated placing over 6,000 cu. yds. of concrete and 250 tons of reinforcement.

Upon completion of the octagon and envelope walls, the internal spider was dismantled. The 'well' formed by the octagon walls was then handed over to the mechanical and electrical engineers for the installation of the reactor vessel.

The reactor vessel, a cylinder about 40 ft. diameter and 60 ft. high, was assembled and welded on site from plates delivered from the works already curved and prepared for welding. A fabrication area was made available adjacent to each reactor (Fig. 10) and the vessel was fabricated in four main sections, each being 40 ft. diameter and weighing over 90 tons. The sections were assembled in line from the reactor so that by using bogies on a single track railway they could be moved to a suitable position under the 100 ton capacity lifting stick.

The 'big stick,' as it became generally known, was located on an 80 foot high steel latticed tower and the head of the stick was guyed back by eight steel ropes to concrete blocks at suitable positions around the site (Fig. 11). The erection of this main lifting

Fig. 12.—An A-Frame support for the Reactor Vessel and 6″ thick Thermal Shielding positioned within the Octagon.

appliance was carried out using more conventional lifting equipment. Three sides of the latticed tower were erected and guyed, then the mast itself was laid at the foot of the open face of the tower and raised into a vertical position within the tower before the remaining face was erected, following which the mast was raised up vertically through the tower. Finally the jib was hoisted, using the mast, and secured to the base of the mast in the usual way. This whole erection operation lasted almost six weeks, but half this length of time was sufficient for dismantling.

The steel soldiers, thermal shield plates and the inverted 'A' frame legs (Fig. 12) which support the reactor vessel were placed within the well of the octagon by means of the big stick and following this operation the finished vessel sections were successively lifted and placed in position. They were then welded together within the octagon to form the complete reactor vessel.

The main task of the big stick was now completed, but it remained in position for use in erecting the 90 ft. high steel ventilation stacks and the Bailey girders over the octagon well. Finally it was used for the erection of the pile cap building and the internal overhead electric crane.

The heat exchangers, four of which surround each reactor, are approximately 18 ft. diameter and 70 ft. high, weighing 180 tons. They were fabricated off site in rings weighing about 25 tons each, being finally assembled and welded in a shop on site provided for that purpose.

The fabricated shells, after testing, were transported by tractor and low loader to a position adjacent to their concrete base pedestals. Here they were transferred to temporary timber beds ready for lifting by a pair of 'gin poles' located on opposite sides of the pedestal (Fig. 13). A belt or saddle was placed round the body of the vessel near to the top and attached to this were the ropes from the gin poles. A tailing guy was attached to the base of the heat exchanger which was then lifted into the vertical position and seated on the pedestal.

At this stage the erection of steelwork had commenced on the adjacent Control Building and Blower Houses. The Control Building is a steel framed and braced structure having six floors including the one at reactor cap level. The floors are chequer plate, filler joist or normal reinforced concrete, depending upon their

Fig. 13.—The Heat Exchanger in position for Lifting.

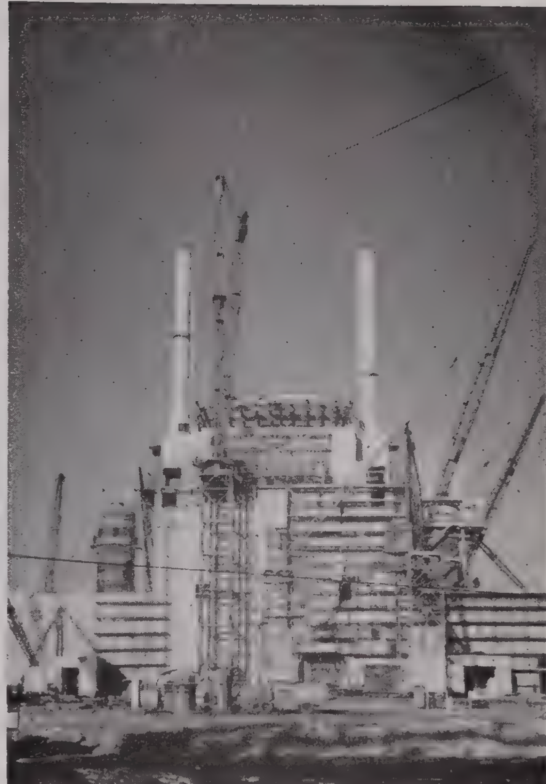

Fig. 14.—The Reactor Building nearing completion.

particular purpose and location. An inner span of 36 feet adjacent to the reactor face carries the main floor area at each level, e.g., control room, switchgear floor, etc., and the outer 14 ft. span houses the staircase, lift well, hoist well and ancillary service rooms.

Steel erection was completed, including the roof steelwork which extended over the area of the main biological shield (Fig. 14). Thus the building could be made weathertight before the more intricate work of concreting the pile cap was put in hand.

The two Blower Houses which, as their name suggests, contain the blowers and fans, etc., for the reactor coolant circuit, are steel framed and braced shed type buildings with welded, solid web, roof girders which span 58 ft. The roof girders were fabricated from M.S. plates and delivered to site in one piece ready for erection. Each building has a height to eaves level of 40 feet and supported on the main columns at a height of 32 feet are gantry girders for an overhead electric travelling crane. The foundations and ground floor slabs of these buildings are complicated by the ducts from the coolant circuits, and large foundation blocks had to be provided for the fans, etc. (Fig. 15).

Meanwhile, following the dismantling of the big stick which had been used for the erection of the reactor vessel, erection proceeded on the Discharge Building. This building surrounds the discharge tube and chamber and is a conventional steel framed building with various floors for operational purposes. Considerable accuracy was required in constructing the floor at the reactor cap level, since the rails for the fuel element discharge machine are set in the reactor cap and continue over to allow the machine

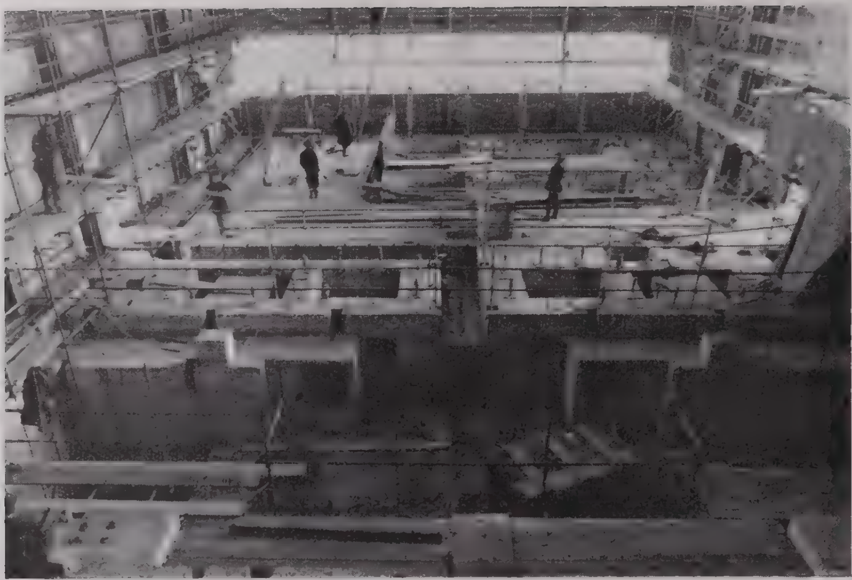

Fig. 15.—Interior of Blower House.

to deliver spent fuel elements down the tube in the discharge building.

Upon the completion and successful testing of the pressure vessel, construction was started on the roof slab forming the biological shield across the top of the octagon and which becomes the operating floor for the charging and discharging of the reactor fuel elements.

This roof slab is penetrated by more than one hundred tubes used for charging the reactor, and the space left between the tubes is very restricted, presenting problems from the design and construction point of view. The slab is 8 feet thick, having a 6 in. thick steel thermal shield suspended from the underside, giving a total weight to be supported across the octagon of about 1,000 tons. The 8 ft. thickness was designed and constructed in four lifts, the lower three lifts including one at 1 ft. thick and two at 2 ft. 4 in. thick respectively. These lifts satisfied the structural requirements and the top 2 ft. 4 in. lift was left to suit requirements not finalised at the time of designing the structural portion. The 1 ft. thick bottom lift was poured in one operation; the second and third lifts of 2 ft. 4 in. were each poured in two areas, first a central area enclosing all the tubes through the roof and then the annulus surrounding this area and bounded by the octagon walls.

When the requirements for the top 2 ft. 4 in. lift were finalised they demanded the accurate setting, to a tolerance in line and level of $\pm\frac{1}{32}$ in., of seven lines of 115 lb. per yard flat bottom rail on which traverse the 50 ton charge and discharge machines (Fig. 16). This portion was constructed in two lifts of concrete; the initial pour, over the full area, being used to position joist stools for the subsequent lining and levelling of the rails, the remainder then being solidly concreted, and having a separate granolithic topping to bring the finished floor up to top of rail level.

The problem with the reactor cap was to support the concrete and thermal shielding during construction. Some method of suspension from above had to be devised, as strutting from the top of the pressure vessel below could not be permitted. To achieve this object two outer sets of 40 ft. span and three inner sets of 50 ft. span double-double Bailey girders were placed at high level over the octagon roof area (Fig. 17). The girders sat on steel cribbages bearing on the main shield walls, with the feet of the cribbages within the depth of the final 2 ft. 4 in. roof lift.

Pairs of rolled steel channels were attached longitudinally under the bottom booms of the Bailey girders. These served to carry movable runway beams running transversely which were used for constructional reasons, but their main purpose was to support 2 in. diameter hanger rods at approximately 5 ft. 4 in. centres along the line of each Bailey girder. The centres of the hanger rods were dictated by the specified centres of the charge tubes which perforate the roof slab.

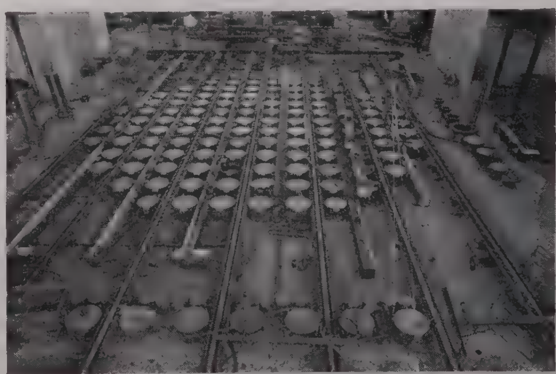

Fig. 16.—Charge and Discharge Machine Rails at Reactor Cap Level.

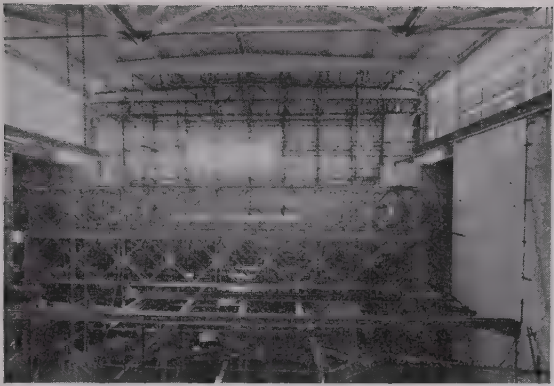

Fig. 17.—Bailey Bridge Assembly.

Fig. 18.—24 ft. Level Floor and Roof Girders—Turbine Hall.

At the lower ends of the rods continuous twin rolled steel angles were attached, to form the permanent seating for the 6 in. thick thermal shield plates. A few inches above these plates further sets of twin angles were provided on which the permanent shutter plates for the concrete roof were placed. A predetermined camber was given to the angles before the plates were welded, so that after concreting the underside of the completed roof would be as near level as possible. The degree of camber was calculated from the results of loading tests which had been carried out on these particular Bailey girders while at ground level, prior to their erection over the roof of the reactor. The camber was achieved by adjustment on the screwed upper ends of the 2 in. diameter rods.

Once the thermal shield plates had been fixed the permanent shutter plates were positioned and seal welded to prevent leakage whilst the concrete was poured. After fixing the necessary tubes the reinforcement was placed and the bottom lift was poured followed by the second and third lifts. Levels were taken after the casting of each lift to check the reduction in camber.

Seven days after pouring the third lift, the concrete having attained two-thirds of its 28 day cube strength, the rods hanging from the Bailey girders were released gradually and systematically, to prevent the possibility of overstress in any individual hanger. Upon release the hangers were burned off just above the concrete surface before the final lift was poured.

A project such as Calder Hall requires ample lifting facilities and initially two 7-ton derrick cranes, with 120 ft. jibs, were provided on tracks at the north and south sides of the reactor buildings. Later, in addition to the 'big stick' already mentioned, 10 ton cranes on 40 ft. high gabbards with tracks, were added on both north and south sides (Figs. 14 and 26). These latter cranes covered the lifting of large valves, duct work, etc., to the higher levels of the carbon-dioxide cooling circuit as well as assisting in the concrete work at these levels and with the erection of the surrounding superstructure steelwork.

The Turbine Hall

The turbine hall is sited centrally between the two reactor buildings and houses 4—23,000 k.w. generators together with their associated steam plant and electrical gear.

The main hall is steel framed and is 240 feet long by 60 feet high with welded roof girders spanning 80 feet (Fig. 18). The roof girders are tapered in plan and in elevation and have a 4-in. camber in the bottom boom. They were prefabricated in two pieces and delivered to the site for final preparation and welding to form the completed girder before erection.

Running the full length of the hall is an internal gantry for an overhead travelling crane, which is supported on the main building columns. At a level of 24 feet above the ground floor slab, a chequer plate access floor is provided around the turbines.

Fig. 19.—Foundations—Turbine Hall.

Fig. 20.—Completed Steam and Alternator Block—Turbine House.

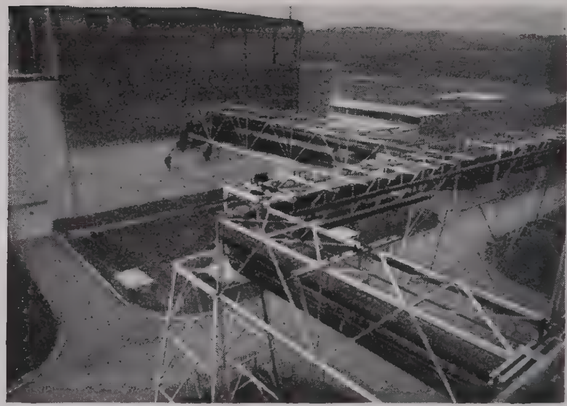

Fig. 21.—Pipe Bridge from Heat Exchangers to Turbine Hall.

Within the area of the hall the foundation work, including ductwork for the cooling water inlet and outlet mains and the turbine foundations, required 2,400 cu. yds. of excavation (Fig. 19).

The turbine foundation block consists of a large reinforced concrete foundation, initially brought up to internal ground floor level, upon which a pre-fabricated steel framework was erected to form a bed at the appropriate level for the turbines (Fig. 19). The framework was then solidly encased in concrete, each block being completed in one continuous pour of 175 cu. yds. for the steam side block and 125 cu. yds. for the alternator block (Fig. 20). Considerable use was made of funnels and flexible delivery tubes during the placing of the concrete and in the early stages of the pouring operation the delivery of the concrete was controlled with the aid of radio communication.

The turbine hall steam annexe is a two-storey steel framed building on the west side of the main hall. During the erection of the steel frame for the main hall the annexe steelwork was omitted so that a 10-ton derrick could be used adjacent to the Hall, proceeding from south to north and then retreating southwards erecting the annexe steelwork as it withdrew.

On the east side of the main hall is another annexe housing the turbine control room and a number of rooms for electrical equipment. After the construction of the cable basement for this annexe the steelwork was erected by means of a steel pole and mobile crane, the foot of the pole being located in the cable basement.

The cable basement is joined to the reactor control buildings by underground cable ducts. These were constructed using reinforced concrete base and roof slabs with brick retaining walls, and incorporated

Fig. 22.—Administration Building.

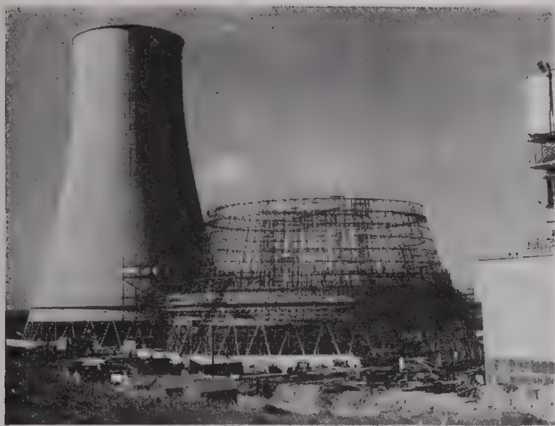

Fig. 23.—Cooling Towers, Calder Hall 'A' Nuclear Power Station.

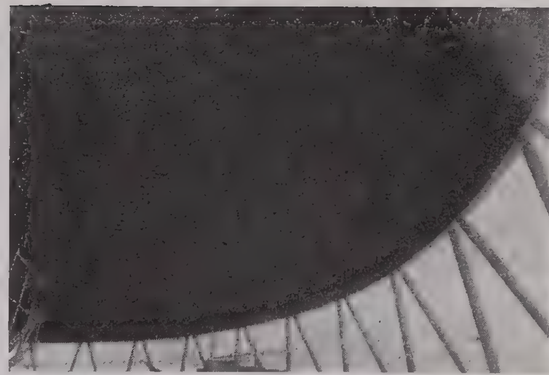

Fig. 24.—Internal Scaffolding arrangement to Cooling Towers.

an internal system of steel racking to carry the electric cables. Due to the considerable amount of work, and plant movement, in the areas adjacent to these ducts, the walls and slabs were designed for traffic loads over the majority of their length.

At the gable ends of the turbine hall, steel framed pipe bridges are used to carry the steam pipes from the heat exchangers (Fig. 21). These are twin lattice girders with horizontal bracing at top boom level, leaving a clear internal space for the erection and maintenance of the pipes.

The Administration Building

On the east side of the turbine hall, and connected to it by a corridor at first floor level, is the administration building for the station (Fig. 22). This is a steel framed service building, part rising to two-storeys, with all the necessary administrative accommodation located on the first floor level. The ground floor is used for maintenance workshops, kitchen and dining rooms. A feature of the administration building is the gable walls which are built in local stone in an attempt to produce harmony with the surrounding countryside.

The Cooling Towers

The two cooling towers (Fig. 23) rise to a height of 300 feet and have a base diameter of 200 feet. Together they are capable of dealing with approximately 150M gallons of circulating water per day, cooling it through 17°F at a rate per tower of 3M gallons per hour.

The shells are supported on 64 columns, each having a diameter of 16 inches and springing from individual foundations below the level of the cooling pond. The columns were cast using semicircular steel shutters which were located at the correct angle by the use of scaffolding.

Fig. 25.—Erection of Services Bridge over River Calder.

Fig. 26.—A Panoramic View of the Project nearing Completion.

The scaffolding for the shell construction was prefabricated from M.S. angles and was supported off the base slab for the pond. As the shell rose so the scaffolding preceded it by a few lifts. The shell was constructed in 94 lifts using timber shuttering on steel clamp arms with special panels to allow for the hyperbolic variation in shape.

In order that construction on the second cooling tower could proceed before the bituminous paint lining to the first shell was completed, the scaffolding was designed in such a way that it could be suspended from the shell at the level of the annular duct, the lower portion then being available for re-use, and so avoiding large scaffolding costs on repetitive work (Fig. 24).

For the external wall of the pond travelling shutters were used, sections of the external wall and internal pond division wall being left out to give access during the construction of the cooling tower shells.

General Services

In any undertaking of the size of Calder Hall the term ' General Services ' usually covers a considerable amount of work. The most interesting item on this part of the project was the Services Bridge over the River Calder joining the new site to the existing Windscale Works (Fig. 25).

The bridge has two cantilever spans located on the Windscale and Calder banks of the river respectively. These have a span of 63 ft. between the tower and trestle and a 21 ft. cantilever over the river to receive a central suspended span of 98 ft., giving a clear distance over the river of 140 ft.

An all welded lattice girder construction was used including site splices, only the horizontal bracing being site bolted. All the steelwork was zinc sprayed. The individual spans were pre-assembled in box form on their respective banks, the Calder side being erected by the use of mobile cranes, and that on the Windscale bank by using steel poles. The central span was assembled on the Calder bank. Under chosen river conditions, with a 30 ton mobile caterpillar crane located in the river at shallow water, it was pushed out along a track suspended between cribbages in the river and then lifted into position by the large mobile crane. It will be appreciated that this operation was more hazardous than has perhaps been made apparent, for the River Calder can rise very rapidly, often as much as seven feet overnight.

Conclusion

The work described covers a more comprehensive field than is normally encountered by the structural engineer in one single project, and it has therefore only been possible to present some of the points of structural interest.

The panoramic presentation (Fig. 26) of Calder Hall ' A ' nearing completion portrays a design and constructional effort which was unique and has provided much valuable experience and information for further advancement of the nuclear power programme.

Acknowledgements

The author wishes to express his thanks to the Managing Director of the United Kingdom Atomic Energy Authority (Development and Engineering Group) for permission to publish this paper, to acknowledge the encouragement given by Mr. T. C. Waters, M.I.Struct.E., Chief Structural Engineer, U.K.A.E.A. (D. & E. Group), and to thank Mr. E. Stone, A.M.I.Struct.E. for the valuable assistance given in the final preparation of proofs prior to publication.

The Design of the Unbraced Stabbogen Arch *

Written Discussion on the paper by Chin Fung Kee,

M.Sc., A.M.I.Struct.E., A.M.I.C.E., A.M.I.C.E.I.

Dr. W. G. Godden commented that the problem of both transverse and axial forces acting on a laterally unbraced arch rib within the range of elastic action raised two important issues, namely, the application of Southwell's method, and the evaluation of stress from an assumed deflection profile.

As the Author had based his work on the use of Southwell's method, Dr. Godden suggested it might be well to consider the range of applicability of that useful device.

Timoshenko[1] showed that if a nominally straight pin-ended strut had an initial eccentricity of y_0 given by

$$y_0 = a_1 \sin\frac{\pi x}{L} + a_2 \sin\frac{2\pi x}{L} + a_3 \sin\frac{3\pi x}{L} + \ldots \qquad (1)$$

then under the action of an axial load H the strut underwent further deformation y_1 due to H given by

$$y_1 = \frac{1}{\left(\frac{H_{cr}}{H} - 1\right)} a_1 \frac{\sin \pi x}{L} + \frac{1}{\left(\frac{4H_{cr}}{H} - 1\right)} a_2 \sin\frac{2\pi x}{L} + \frac{1}{\left(\frac{9H_{cr}}{H} - 1\right)} a_3 \sin\frac{3\pi x}{\mu} + \ldots \qquad (2)$$

where H_{cr} was the first critical load or Euler load of the strut.

It would be noted that in (1) the terms of the series represented the basic modes of failure in the strut, and in (2) the values H_{cr}, $4H_{cr}$, and $9H_{cr}$ etc., were the corresponding critical loads.

Southwell's method in effect neglected all terms except the first in (1) and (2) whence

$$y_1 = \frac{y_0}{\left(\frac{H_{cr}}{H} - 1\right)} \qquad (3)$$

and the axis transformation used in the Southwell method resulted in a linear relationship between $\frac{y_1}{H}$ and y_1, from which an accurate extrapolated estimate for H_{cr} could be found.

It was important to remember this assumption of the higher terms of (1) and (2) being negligible before applying it to any specific problem, as for the assumption to be valid the following conditions must be fulfilled :—

(1) The critical loads of the system must be well separated.

In the Euler strut problem this was so, the ratio between the first three critical loads being 1 : 4 : 9. Hence as the applied H approached the primary critical load H_{cr} the first term in (2) became very large relative to the rest. It could, however, be seen that if for any reason the critical loads were not so well separated, the solution was not so simple—for example, if the second critical load was close to the first, then the second term would increase rapidly with the first causing a break-down in the assumption.

In a laterally unbraced arch made with a tension membrane, the first two critical loads were almost coincident[2], and although experimentally one could get round this difficulty by measuring the deflection at mid-span where the even terms vanish, the relative value of the next critical load, namely the third, was not known.

(2) The values of H used must not be small relative to H_{cr}.

This could be seen from (2). Only as H approached H_{cr} did the first term become so large that the others could be neglected. In practice the Southwell Plot seemed to work well for values of H between $0{\cdot}5$ and $1{\cdot}0\ H_{cr}$, below $0{\cdot}5\ H_{cr}$ experimental points might not always lie on the linear part of the graph. This hardly mattered when the value of H_{cr} was required, but could be serious if any effort was made to use the method for predicting deflections for low values of H. For example in Figure 2, graphs *f*, *e*, and *d* were plotted for high values of H that might be outside the design range, and whereas the inverse slope of this would give an accurate estimate for H_{cr}, the designer when estimating stress might be interested in what happened in the lower part of the graph, and as he had explained, it was problematical if the points would lie on the drawn straight line in that range.

(3). If one dealt within the range of say $H = 0$ to $0{\cdot}5H_{cr}$, then the Southwell Plot would only be linear if the second and higher terms of (1) were negligible—that is, if the initial eccentricity of the system (whether due to construction or wind loading) approximated very closely to the primary buckling mode. This must be watched carefully as it was not always close enough. In the case of an arch rib the deflection form due to uniform transverse loading was not quite the same as the primary buckling mode (perhaps due to torsional effects). Hence if the higher terms in the series were neglected, there might be an appreciable error in estimating the value of z_1 the increase of z due to H.

It would appear thus that when dealing with loads of less than say 50 per cent of the buckling load, it was not safe to assume that the Southwell Plot was a valid means of calculating displacement unless one had some knowledge of the form of the eccentricity curve (1) and also of the series equivalent to (2) in the problem in hand.

With regard to the evaluation of maximum bending stress from a knowledge of the central deflection,

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the expression

$$z = k\left(\sin\frac{\pi s}{S} - \frac{1}{3}\sin\frac{3\pi s}{S}\right) \quad . \quad . \quad . \quad (4)$$

was an assumed primary buckling mode for a perfectly plane arch, that was to say, it was equivalent to the first term of (1) for a straight strut. This expression had been shown to be accurate enough for an energy solution of the first critical load, but that solution does not depend on taking $M = \frac{d^2z}{ds^2}$ which could be inaccurate, but on evaluating M from the external forces acting on the system. It was not necessarily justifiable however to take this assumed expression, (or the even more approximate equivalent in x instead of s) and differentiate it twice thus multiplying errors, unless the expression was known to be very accurate. Also, the two causes of initial eccentricity, namely, construction tolerances and wind could not always be taken in the form of (4) which could be the only real justification of this method of estimating M and hence maximum stress. It would appear that a safer method for finding maximum moment was from a consideration of the external forces acting on the structure in any instance, and that required a knowledge not only of central deflection, but also of the overall deflected shape taking all the important factors, including the torsional rigidity into account.

Dr. Godden considered Mr. Chin had made an interesting start into what was an extremely difficult problem of elastic analysis, and the precise data of Figure 2 showed clearly the behaviour of a solid circular section arch rib deflecting at mid-span up to almost twice its own width under the action of large axial forces. He thought, however it might be unwise, to infer from limited data that arches of any section would behave in this way over the full range of loading, and Dr. Godden felt that to use the method advocated in the Paper for estimating maximum stress might possibly lead to considerable error in some cases, especially when the amount of transverse deflection was small.

References

1. Timoshenko, S., "Theory of Elastic Stability." p. 32.
2. Godden, W. G., "The Lateral Buckling of Tied-Arches." *Proc.Inst Civil Engs.* Pt. III. August 1954.

Mr. M. Gregory was interested to see the Southwell Plot on deflections used to obtain not only the elastic critical load of the arch but also the effect of practical imperfections, in order to establish empirical information useful in design. Some experimenters had used the plot merely to confirm the values of critical loads calculated by theoretical means, but the power of the plot in obtaining quantitative data concerning the effects of "imperfections" such as crookedness, eccentricity, or lateral loading, had been largely neglected. It was the collapse load that interested the designer, and it was pleasing to note that Mr. Chin had made good use of all the data obtained from the equation of the linear deflection plot in order to predict the maximum stress in the arch rib. Collapse could be assumed to occur when the yield stress was reached, as the reserve strength in the plastic state was probably small.

Mr. Gregory then referred to the Southwell Plot on measured strains. He had found this plot to be linear for many buckling problems and the equation of the plot led directly to a design criterion. Strains were conveniently measured and the deflection analysis avoided. It would be interesting to see the results of applying this method to the unbraced arch rib.

References

Gregory, M. S., "The Use of Measured Strains to obtain the Critical Load of a Strut," and "The Use of Measured Strains to obtain the Critical Load of a Triangular Frame." *Civil Engineering,* London, October 1959. "Further Research on Bolted Angle Struts" and "The Collapse of Triangulated Frames containing Bolted Angle Struts." *Water Power,* August and October 1959. "The Use of the Southwell Plot on Strains to Determine the Failure Load of a Lattice Girder when Lateral Buckling Occurs." *Australian Journal of Applied Science,* Vol. 10. No. 4. December 1959.

The Author in reply said there was no doubt from Timoshenko's analysis that in the case of the pin-ended strut, the values of H must be a large fraction of H_{cr} before the Southwell plot would yield a close estimation of the critical load and that with low values of H, the points might not lie on the linear part of the plot. Unfortunately, a similar expression as equation (2) was not available for the Stabbogen arch, and it would be unjustifiable to infer from this equation that the arch would also behave in the same manner. Dr. Godden had indicated that in the strut problem, the ratio of the first two critical loads was 1 : 4, whereas in the case of the arch these two critical loads were almost coincident. That was evidence that equation (2) could not be applied to the arch in toto. For want of a series equivalent to this equation for the strut, it would be more prudent and justifiable to deduce from experimental behaviour.

Experiment No. 3 (f) was performed over values of H ranging from $0{\cdot}22H_{cr}$ to $0{\cdot}85H_{cr}$. The experimental data were as given below :

Table 1.

$H_{cr} = 101{\cdot}1$ *lb.*

$f = 0{\cdot}18$

H (*lb.*)	z (*cm*)	$(z/H) \times 10^3$
O	O	O
22·25	0·11	4·94
38·8	0·26	6·70
52·5	0·46	8·76
58·8	0·59	10·02
61·7	0·69	11·18
65·5	0·79	12·06
68·8	0·91	13·22
72·0	1·05	14·60
75·2	1·21	16·10
78·4	1·45	18·50
80·6	1·69	20·96
82·4	1·93	23·42
84·2	2·29	27·20
86·1	2·75	31·96
87·3	3·53	40·40
87·1	4·01	46·00
86·9	4·21	48·40
86·55	4·43	51·20
86·3	4·61	53·40

The plot of the results in the elastic range was given in Figure 6. It was apparent that over the entire range of H of from $0{\cdot}22H_{cr}$ to $0{\cdot}85H_{cr}$ the points were consistent and lay close to the straight line. The intercept on the z axis of the Southwell plot would still have the same value, irrespective of whether H was a small or a large fraction of H_{cr}.

Experiments with small values of H should be performed with special care. The existing design of the testing bench did not prevent the occurrence of local instability or slip at the arch bearing plates (J in Plate 2) resulting from the shortening of the chord length

due to lateral deformation under a transverse load. This movement of the ends of the arch would produce a scatter which it would be incorrect to attribute to behaviour predicted by equation (2) for the strut. It was with the intention to exclude such local instability that experiments such as No. 7 were carried out with such high initial values of H.

The Author referred to Dr. Goddens' statement that since the deflected form due to initial curvature and to lateral wind load could not always be taken as that of the characteristic buckling mode, there was no real justification in basing the evaluation of maximum bending stress on the characteristic buckling mode, and Mr. Chin suggested that to assess the validity of this statement, one might first bear in mind that the theoretical value of H_{cr} was evaluated on the energy criterion of stability, i.e. Raleigh's Method—viz.: that at the critical load the work done by the external forces was equal to the strain energy acquired by the system. The strain energy due to bending and to torque were taken into account; the expression for bending moment and for torque were based on the buckling mode given by equation (4). Southwell (2) proved that in assessing the critical load of the pin-ended strut by Raleigh's Method, a value very close to the critical load would still result even if the deflected form assumed was not quite that of the characteristic buckling mode. In the case of the arch, both Dr. Godden and the Author had found by experiment that the Southwell plot would still yield a close estimation of the critical load even though the initial curvature and deflected form of the arch were nowhere close to that given in equation (4). In fact, "Even in the extreme cases where the maximum displacement occurred at the hanger adjacent to that at mid-span, this lack of symmetry had no apparent effect on the value of the crippling load obtained." (3). Though the deflected form due to a transverse load was not identical with the characteristic buckling mode, the critical load obtained from the Southwell plot was in close agreement with the theoretical value of H_{cr}, as was evident from the results for experimental series 3 and 7 quoted in the Paper.

If the difference between the value of the critical load under an applied lateral loading and that computed on the basis of the characteristic buckling mode was small, then the error in assuming that the expressions for bending moment and for torque for the arch under the applied lateral load were the same as those derived for an arch distorting under the characteristic mode was even smaller. That was so because the total strain energy was obtained by integrating the square of the expressions for bending and for torque. Therefore the method which the Author had presented for the evaluation of stress from the deflection profile was not inadmissible.

He pointed out further that in those experiments where the arches were loaded until collapse, there was close agreement between the collapse load registered and the maximum load H_m calculated by the method outlined in the Paper (Table 2)[1].

As would be seen from Table 1, experiment 3 (f) was one such experiment. The value of e was obtained from the Southwell plot (Figure 6) and used in the calculating H_m.

Table 2.[1]

Experiment	f	e-cm (Obtained from Southwell plot)	*Theoretical Value of Hm* (lb)	*Maximum load* registered in experiment (lb)
3 (f)	0·18	0·40	90·0	87·3
2 (f)	0·16	0·28	98·7	98·8

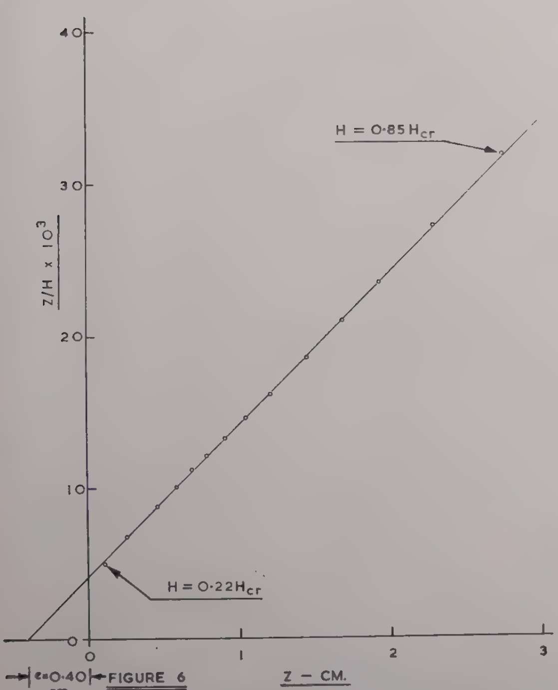

FIGURE 6

SOUTHWELL PLOT— EXPERIMENT NO. 3(f)

THE AUTHOR pointed out that experiment 3 (f) was performed with a lateral load in the form of a point load applied at the crest of the arch rib. The deflected form under this form of loading was obviously not the same as the characteristic buckling mode given in equation (4), but nevertheless the maximum load that the arch could sustain under test was in close agreement with the calculated value. That was clear indication that the doubts raised by Dr. Godden were unfounded.

Experimental series No. 7. was conducted with an arch of circular cross-section and it was not intended that the results obtained would be applicable to arches of all cross-sections. Figure 5 with its curves for sections of various γ, was mainly to enable the designer to calculate the value of H_{cr} and to note the very small effect which different values of γ had on the critical load.

Mr. Gregory's method of assessing critical loads by measuring strains was interesting and certainly merited the attention of future researchers on the arch problems and for verification of some of the issues raised in the Paper and in the discussion. The Author hoped that his observations might be of assistance to future investigators in the same way as those made by Dr. Godden and others on this subject, had been of value to him.

References

1. Chin Fung Kee. "The Inelastic Buckling of Arches," M.Sc., thesis, Queen's University, Belfast, 1953.
2. Southwell, R. V., "Theory of Elasticity"—1941.
3. Godden, W. G., "The Lateral Buckling of Tied Arches," Part III, *Proc. I.C.E.*, August, 1954.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 23rd June at 5 p.m. Mr. L. E. Kent, B.Sc. (Eng.), M.I.Struct.E., M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

ARTHUR, Thomas Euinson, of Belfast, Northern Ireland.
BOUGHTON, Brian William, of Ilford, Essex.
BRIGHTON, John Henry, of Hillingdon, Middlesex.
CAMPBELL, Colin Neil, of Wanganui, New Zealand.
CHATTERJEE, Prabhat Kumar, B.Sc., of London.
CHEONG KWAI HIN, of Singapore.
DAS, Bishnu Pada, B.Sc., of London.
GARRETT, Ian John, of Wellington, New Zealand.
JUCKES, Douglas Phillip, of London.
LAI SHUI KWAN, of Hong Kong.
LEWIS, Leslie Peter, of London.
LIM CHIN TIAN, of Singapore.
MITCHELL, John Michael, of London.
NWODILI, Michael Onukwue, of Manchester.
POLYCARPOU, Savvas, of London.
ROBERTSON, Richard, of Glasgow.
SCANDLING, John Michael, of Bristol.
TANG HUNG CHUN, of Hong Kong.

Graduates

ALSOP, David John Andrew, B.Eng., of Billingham-on-Tees, Co. Durham.
BHATTACHARJEE, Ashimanada, B.Sc., of Birmingham.
CHADWICK, Neil, of Salisbury, Southern Rhodesia.
CHEN DZU BIAO, B.Sc., of London.
CHEUNG HE-CHOI, of Kowloon, Hong Kong.
CHIANG, Robert Sheng Cheng, B.Sc. (Eng)., of Hong Kong.
DOUET, Louis Maurice, of London.
EL ABD, Mohamed Galal, B.Sc., of Cairo, Egypt.
EMMETT, Michael John, of London.
FARMER, Peter Michael, B.Sc., of Stoneygate, Leicester.
FLEMING-BROWN, David Hugh, M.Sc., of East Kilbride.
GARNETT, Eric William, of Culcheth, Nr. Warrington, Lancs.
GHOSH, Karunamoy, B.Eng., of Calcutta, India.
GORADIA, Thakordas Andarji, B.E., of Littleover, Derby.
HALLAHAN, Michael Lawrence, of Brisbane, Queensland, Australia.
JOWETT, Arnold, of Leeds.
LANE, Victor Percy, B.Eng., A.M.I.C.E., of Liverpool.
LI KUI WAI, B.Sc.(Eng.), of Hong Kong.
MCCONNELL, John, of Musselburgh, Scotland.
MCKAY, Andrew, of Lanarkshire, Scotland.
POPO-OLA, Oladiran Nathaniel, B.Sc.(Eng.), of London.
QUINN, Felim, B.Sc., of Newry, Co. Down.
RAGHAVEN, Nemam Echambady Vijaya, B.E., of Calcutta, India.
ROGERS, Brian, of Darlington, Co. Durham.
ROY, Subash Chandra, of Delft, Netherlands.
SHILLING, Michael Frank, of Greenford, Middlesex.
SILLETT, Donald Frank, of London.
SPENCER, Stanley Cowan, of Manchester.
SUBRAMANIAM, Thuryrajah, of Singapore.
SUNDARAM, S. K., B.Sc., of Kerala State, India.
TAYLOR, John, of Hayes, Middlesex.
TSOU CHEN CHUNG, Cyril, B.Sc., of Glasgow.
WEBSTER, William Gould, of Gerrards Cross, Bucks.

Associate-Members

CULLIMORE, Macdonald Stuart George, B.Sc., Ph.D., of Bristol.
KHADILKAR, Bhaskar Shrikrishna, B.E., of Bombay, India.
NEVILLE, Adam Matthew, M.Sc., Ph.D., A.M.I.C.E., of Hale, Cheshire.

Members

ANDREWS, Denis Adrian, M.I.C.E., of London.
MEDCALF, John Deverell, B.A., B.A.I., of Dublin, Ireland.
ROWE, Richard Harry, of North Shields, Northumberland.
RUSSELL, Peter, B.Sc., M.I.C.E., of Edinburgh.

Transfers

Students to Graduates

DESMOND, Howard George Peter, of Cardiff, Glam.
LAM TIN SANG, of Hong Kong.
NG SAI-WAH, of Holmcroft, Middlesex.
ROBINSON, Michael Douglas, of Newton, Chester.
SCOBLING, Michael James Albert, of Hampton, Middlesex.
THOMPSON, Nigel Cooper, of London.
WILLCOCKS, Anthony John, of Woking, Surrey.

Graduates to Associate-Members

ANNELLS, Michael, of Singapore.
COX, Alan David, of Birmingham.
CRAWFORD, Robert Sinclair, of Huyton-with-Roby, Lancs.
GABAY, Leslie Oliver, ST. C. of Twickenham, Middlesex.
GOODWIN, Donald John, of Bradford, Yorkshire.
HARES, Malcolm John Edmond, of Blaenavon, Mon.
HARRIS, Edward George, of London.
HEWSON, Kenneth Norman, B.Sc.(Eng.), of Transvaal, South Africa.
HOMERSHAM, Peter Gerald, of Romford, Essex.
JONES, Michael Sidney, of Battlefields, Shrewsbury.
MACLACHLAN, Ian Hamilton, of West Montreal, Canada.
PESKETT, George John, of London.
SHAH, Chandrakant Bhailal, of Ahmedabad, Bombay State, India.
WILKINSON, Frederick Alan, of Bolton, Lancs.

Associate-Members to Members

BISHOP, Reginald Walter, O.B.E., B.Sc.(Eng.), M.I.C.E., of Pinner, Middlesex.
BOOTH, Clifford Stanley, A.M.I.C.E., of Bolton, Lancs.
DUCKWORTH, Norman, of Salford, Lancashire.
EASTWOOD, Wilfred, B.Eng., Ph.D., A.M.I.C.E., of Sheffield.
GIDWANI, Govind Menghraj, B.E., M.Sc., A.M.I.C.E., of Warlingham, Surrey.
HOLT, Hermann Paul, of London.
LINTILL, Walter Henry, of Cardiff.

LONG, Alwyn Edward, B.Sc.(Eng.), A.M.I.C.E., of New Milton, Hants.
MARCHAM, Albert, B.Sc.(Hons.), A.M.I.C.E., of Durban, South Africa.
MARSHALL, George, of Bulawayo, Southern Rhodesia.
NEWTON, Peter Dennis, B.Sc., A.M.I.C.E., of Middlesbrough, Yorks.
VISWESWARA RAO, Josyula, B.E., M.Sc., of Kharagpur, India.
WATTS, William Sanderson, A.M.I.C.E., of Culcheth, Nr. Warrington, Lancs.
WILLIAMS, John Olav, B.Sc., A.M.I.C.E., of Liverpool.

Member to Retired Member

BROWN, Robert Cyril Woodlands, of Douglas, Isle of Man.

OBITUARY

The Council regret to announce the deaths of Edgar BAIN, Hector Christopher BENNETT, John Percival CLARK, and Arthur John HOPE (Members); Alexander Somerled MACKICHAN (Retired Member), William Archibald DUROSE (Retired Associate Member) and Peter HARRIS (Student Member).

RESIGNATION

Notification was given that the Council had accepted with regret the resignation of Michael William PICKARD, (Student Member).

HONOURS AWARD

In offering their sincere congratulations to the following member on the distinction conferred upon him, the Council feel that they are also expressing the good wishes of the Institution.

ORDER OF THE BRITISH EMPIRE—O.B.E. to Mr. R. H. L. Sung (Member).

THE MAITLAND LECTURE

The Council having considered the entries for the Maitland Lecture Competition for 1960-61, have decided to make no award. The regulations governing the award have been revised and are now as follows :—

(1) The Institution of Structural Engineers shall include once every two years in their Sessional Programme of Meetings a written lecture to be known as " THE MAITLAND LECTURE " commencing with the session 1962-1963.

(2) The Maitland Lecture Committee shall submit a list of suitable names to the Council of possible lecturers and the Council will thereafter invite one of these distinguished personages, not necessarily an Engineer, to deliver the Maitland Lecture. The subject of the lecture is to be selected by the lecturer and approved by the Council.

(3) Invitations to the lecturer shall be timed so as to ensure that not less than 18 months is available to the lecturer for the preparation and submission of the lecture, which will be given at an Ordinary Meeting of the Institution in the early part of the Session.

(4) No lecture shall have been published or read elsewhere. It will, in due course, be printed in the Journal.

(5) The lecturer shall receive a Maitland Silver Medal, suitably inscribed, and in addition, a premium to the value of £250.

(6) The Maitland Lecture Committee shall be constituted annually and shall consist of the President and three of the immediate Past Presidents.

EXAMINATIONS, JANUARY, 1961

The Examinations of the Institution will next be held in the United Kingdom and overseas on 10th and 11th of January, 1961, (Graduateship,) and 12th and 13th January, 1961 (Associate-Membership).

RULES OF CONDUCT
UNION OF SOUTH AFRICA

Under Clause 3 of the Institution's Rules of Conduct a corporate member regularly practising in some part of the Commonwealth other than the United Kingdom may order his conduct according to the Rules of Professional Conduct or Code of Ethics of any national society or body in that part of the Commonwealth and recognised for this purpose by the Council.

Notice is hereby given that the Council have officially recognised the Rules of Conduct of the South African Institution of Civil Engineers for this purpose.

ANNUAL DINNER, 1961

The Annual Dinner of the Institution will be held at the Dorchester Hotel on Friday, 5th May, 1961.

EXAMINATIONS

PREPARATION FOR THE EXAMINATIONS OF THE INSTITUTION BY ATTENDANCE AT TECHNICAL COLLEGES

A candidate for Graduateship or Associate-Membership may be able to attend a Technical College; these notes are intended to guide him in choosing the most suitable instruction.

PREPARATION FOR THE GRADUATESHIP EXAMINATION—

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the Candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At Technical Colleges, courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

PREPARATION FOR THE ASSOCIATE-MEMBERSHIP EXAMINATION

At some Technical Colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering. These cover only part of the requirements for the Associate-Membership Examination.

Colleges in List 'A' provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

INTERNATIONAL ASSOCIATION FOR SHELL STRUCTURES

As a result of the rapid development in the design and construction of Shell Structures, an International Association has been formed to organise meetings and congresses for the interchange of ideas on the subject.

The Association held its First International Colloquium in September last, and more than 100 specialists from twenty countries attended the meetings.

Further information may be obtained from the International Association for Shell Structures, Alfonso XII, 3, Madrid (7), Spain.

REPRESENTATION

The Council have appointed the following Institution representatives :

British Standards Institution—
Council for Codes of Practice :
Mr. L. E. Kent (President) and Mr. B. Scruby (Member of Council) to succeed Mr. Walter C. Andrews (Past President) and Mr. Gower B. R. Pimm (Past President).

Asbestos Cement Products Industry Standards Committee ASB/ :
Mr. C. E. Cannons (Member), to succeed Mr. P. G. Bowie (Member).

*Ready-Mixed Concrete, Technical Committee CEB/*9 :
Mr. J. A. Derrington (Associate-Member), to succeed Mr. P. G. Bowie (Member).

Proposed Code of Practice on Prevention of Steel Corrosion—
Drafting Committee :
Mr. R. W. Schofield (Member).

Revision of CP 101—*Foundations and sub-structures for houses, flats and schools of not more than two storeys. Drafting Committee* :
Mr. R. D. McMeekin (Member).

Revision of CP 204—*Insitu Flooring, Drafting Committee* :
Mr. S. B. Tietz (Associate Member).

Revision of CP 143.201 : 1951 — *Asbestos-Cement Sheet Roof Coverings, Drafting Committee* :
Mr. A. W. Hill (Member)

CP 112—*The Structural Use of Timber, Drafting Committee* :
Mr. J. H. Japp (Associate Member).

Joint Committee on Materials and their Testing :
Mr. L. E. Ward (Member of Council), to succeed Mr. J. Singleton-Green.

BSI Codes of Practice Scottish Advisory Committee—
Mr. E. H. Cooley (Associate-Member)
Mr. W. Shearer Smith (Member).

BSI Building and Engineering Services Committee in Scotland, B/—/2 :
Mr. W. T. Taylor (Associate-Member).

Darlington College of Further Education—Mechanical and Production Engineering Advisory Sub-Committee :
Mr. B. K. Mayoss (Associate Member) (re-appointment).

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary : W. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs.

MIDLAND COUNTIES BRANCH

Hon. Secretary : S. M. Cooper, A.M.I.Struct.E., "Applegarth," Hyperion Road, Stourton, Nr. Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

Hon. Secretary : P. D. Newton, B.Sc., M.I.Struct.E., A.M.I.C.E., c/o Cusson & Partners, 112, Borough Road, Middlesbrough, Yorkshire.

NORTHERN IRELAND BRANCH

Hon. Secretary : L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast, 5, Northern Ireland.

SCOTTISH BRANCH

Hon. Secretary : W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTH WESTERN SECTION

Hon. Secretary : C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : W. D. Hollyman, 41, Greensfield Avenue, Dinas Power, Glamorgan.

WESTERN COUNTIES BRANCH

Hon. Secretary : A. C. Hughes, M.Eng., A.M.I.Struct.E., A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.Struct.E., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During weekdays, Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary : J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary : R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Chairman : R. A. Sutcliffe, M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

Hon. Secretary : K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

NIGERIAN SECTION

Chairman : J. W. Henderson, E.R.D., B.Sc., M.I.Struct.E., M.I.C.E.

Hon. Secretary : A. Brimer, A.M.I.Struct.E., Brimer, Andrews and Nachshen, Private Bag Mail 2295, Lagos, Nigeria.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Chairman : T. F. Lee, B.Sc.

Hon. Secretary : W. N. Cursiter, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Redpath Brown & Co. Ltd., P.O. Box 648, Singapore.

OFFICIAL APPOINTMENTS

Air Ministry Works Designs Branch requires in London Structural Engineering, Designer Draughtsmen for reinforced concrete or structural steelwork of all types. Applicants must have adequate training and several years experience. Some site experience and possession of recognised technical qualification an advantage. Financial assistance and time off may be allowed for recognised courses of study. Promotion and pension prospects. Five day week with eighteen days paid leave per year initially. Salary ranges from £680 (at age 25) to £900 p.a. Commencing salary dependant on age, qualifications and experience. Applicants, who must be natural born British subjects, should write to Air Ministry, W.G.d., Lacon House, Theobalds Road, London, W.C.1, or apply to any employment Exchange quoting King's Cross 3745, giving age, details of training, qualifications, full particulars of former posts held and copies of any testimonials. Candidates selected will normally be interviewed in London and certain expenses reimbursed.

ELECTRICITY SUPPLY BOARD (Dublin)—*Vacancies for Structural Design Engineers (Temporary)*. The Electricity Supply Board has vacancies on its Civil Engineering Design staff at its Head Office in Dublin for (a) Engineers not over 35 years of age with at least eight years experience on the design and construction of civil engineering work, the major portion of which should be on heavy reinforced concrete and/or structural steel work ; salary will be not less than £1,009 p.a. in a Scale rising by annual increments to a maximum of £1,257 p.a. (b) Engineers not over 30 years of age with at least four years experience as set out above for (a), salary will be not less than £817 p.a. in a Scale rising by annual increments to £1,162 p.a. Candidates must possess a University degree in Civil Engineering or an equivalent qualification. Design experience on steam generating stations would be an advantage. The commencing salaries within the above scales will depend on qualifications and the nature and extent of experience, but will be not less than the figures mentioned above. The appointments will be on a temporary basis and are expected to continue for at least three years. Applicants should clearly indicate the particular position(s) in which they are interested ; applications should set out the candidates' age and full details of their qualifications and experience (with dates) and should reach the Personnel Officer, 27, Lower Fitzwilliam Street, Dublin, not later than fourteen days from the date of this publication.—J. G. Gargan, *Secretary*.

STRUCTURAL ENGINEERS

required in Architect's Department, L.C.C., salary up to £1,250, in :

(a) *Structural Engineering Division*.—Extensive programme includes multi-storey flats, schools, offices, warehouses and other buildings.

(b) *Structural Engineering Division*.—To deal with applications in respect of special structures under Part IV of the London Building Act (A) Act, 1939, and the constructional Byelaws. Interesting work involving consideration of unconventional forms of construction.

(c) *District Surveyors' Service*.—Work mainly outside involving negotiations with architects, engineers and surveyors and supervision of works in progress.

Good prospects in all these aspects of the Council's work. Form and particulars from Hubert Bennett, F.R.I.B.A., Architect to Council (EK/SE/1638/8), County Hall, S.E.1.

SURREY COUNTY COUNCIL.—Applications invited for appointment of Structural Engineers, salary rising to a maximum of £1,220 p.a., according to age, qualifications and experience, plus £40 to £45 p.a. London Allowance. Must be qualified Structural or Civil Engineer and have had experience in design and detailing of steelwork and/or reinforced concrete for medium to large scale contracts. Applications stating age, qualifications, experience and present salary, accompanied by copies of three testimonials, to County Architect, County Hall, Kingston, as soon as possible.

PLYMOUTH and Devonport Technical College, Tavistock Road, Plymouth.—Principal, E. Bailey, B.Sc., F.R.I.C., A.M.I.Chem.E.; Full-time courses in civil engineering, Session 1960-61 ; B.Sc.(Engineering) London External, College Diploma, Course of three years duration ; Recognised by I.C.E. as exempting holders from parts I and II of the Associateship Examination. For full particulars apply to the College Registrar, Clerk to the Governors.

SITUATIONS VACANT

CONSULTING Engineers, require Designer/Detailer for R.C. and Steel framework, retaining walls, foundations, piling, etc. Write giving full details of training, experience, qualifications and salary required to, Hurst, Peirce & Malcolm, Artillery House, Artillery Row, Westminster, S.W.1.

CAXTON REINFORCED CONCRETE LTD.

Applications are invited for the following progressive and permanent appointments to our Engineering Staff for a large and varied programme of reinforced concrete work.

(a) *Senior Designers*.—Min. qualifications H.N.C., with some experience in design and a good knowledge of structures. Must be capable of handling contracts from start to finish without supervision.

(b) *Junior Engineers*.—Min. qualifications O.N.C. capable of detailing and able to design from outline schemes.

Pleasant working conditions, good salaries, superannuation scheme, usual holidays.

Please write to Chief Engineer, 45, Bedford Row, W.C.1, giving brief details of qualifications, positions held and present salary.

CONSULTING Engineers require in their London office experienced R.C. designers and designer/detailers ; also experienced detailers willing to learn design. Five day week, good salaries for the right people. Apply in writing to S. Zukas, Chief Engineer, John de Bremaeker & Partners, 3, Southampton Place, London, W.C.1.

EDWARD A. PITCHER & Partners require the following staff for their new E.C.4 offices : Section Leader for R.C. Dept., R.C. designers, R.C. detailers, drawing office Junior. Good salaries in accordance with experience and qualifications. Luncheon vouchers, holidays honoured. Apply 14, Queenhithe, E.C.4.

E. J. COOK & Co. (Engineers) Ltd., require design engineers and designer/detailers for reinforced and prestressed concrete. Salaries in accordance to ability and experience. Five day week. Luncheon vouchers ; Pension scheme ; Holiday arrangements will be honoured. Write or telephone for application forms from 54, South Side, Clapham Common, S.W.4. (Macaulay 5522).

E. J. COOK & Co. (Engineers) Ltd., have vacancy for designer/detailer in Structural steelwork office, with opportunity to gain experience in reinforced concrete work. Salary according to experience and ability. Five day week. Luncheon vouchers ; Pension scheme ; Holiday arrangements will be honoured. Apply to 54, South Side, Clapham Common, S.W.4. (Macaulay 5522).

CLASSIFIED ADVERTISEMENTS—continued

EXPERIENCED Designer/Draughtsman with a knowledge of precast and/or insitu work, is required for the Design office (near Stockport) of a nationally established firm of reinforced concrete specialists. The successful applicant will be expected to work under a minimum supervision on a wide range of designs, both in the project and final design stages. Initiative and hard work will be rewarded. There is a contributory superannuation scheme in force, and existing holiday arrangements will be honoured. Applications stating salary required to Box 9103, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

EXPERIENCED Senior and Junior Structural Draughtsmen required by Structural Engineers in Harrogate. Good working conditions—canteen facilities—pension scheme. Apply in own handwriting, stating age, qualifications and salary required to Chief Draughtsman, Octavius Atkinson & Sons Ltd., Prospect Works, Starbeck, Harrogate.

HOLLOWAYS

HOLLOWAY BROTHERS (LONDON) LTD.

have a vacancy in their Building Division for an

ASSISTANT STRUCTURAL ENGINEER

Applicants should be aged 25 to 35 and have experience in design and detailing of reinforced concrete and steel structures. A knowledge of building construction and contracting work would be an advantage. Applicants should preferably be qualified Structural Engineers.

The Company offers five day week, three weeks annual holiday and operates a contributory Pension Scheme.

Applications giving details of age, experience and qualifications should be sent to

THE SECRETARY
157, Millbank, London, S.W.I.

HUSBAND & Co. Consulting Engineers, have several vacancies for designer/detailers, experienced in reinforced concrete and steel, preferably A.M.I.Struct.E. Salary according to qualifications and experience, but not less than £900 p.a. Luncheon vouchers are provided and there is a pension scheme. Apply in writing to Husband & Co., 58, Victoria Street, London, S.W.1.

J. H. COOMBS & Partners have vacancies in their London and Sunbury offices for senior designer draughtsmen fully experienced in multi-storey R.C. framed buildings and capable of handling a large contract from start to finish. Vacancies also exist for good detailers. Write, stating age, experience and salary required to Thames Corner, Sunbury-on-Thames, Middlesex.

CLASSIFIED ADVERTISEMENTS

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in 1960

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We hope that these men will help to maintain Truscon's own tradition of quality—and this applies *both* to construction *and* design. Indeed it may be that the early collaboration between our design and construction staff together with the constant interchange of information between office and site which an organisation such as Truscon permits, will of itself prove an attraction to the ambitious and imaginative, yet experienced, men we are seeking for our design offices.

The following positions are vacant:

London

2 reinforced concrete design engineers, with first-class experience of both project schemes and working drawings.

2 reinforced concrete detailers, again with first-class experience.

Harlow, Birmingham and Manchester

In each office: one designer and one detailer with similar qualifications to the above.

Please write initially to our Director-Engineering at Truscon House, Lower Marsh, London, S.E.1: giving brief details of your qualifications, positions held, experience gained and salaries earned. We are anxious to give applicants every opportunity to justify themselves at an interview, which will be in confidence and can, if necessary, be arranged outside business hours.

Jobs with Truscon are, subject to usual conditions of employment, permanent and secure yet progressive. Salaries are good, as are holidays and sick pay entitlement. We have a good pension scheme, with dependants' benefits, and a profit sharing scheme for staff. Promotion is by merit alone, and although we already have a number of mature young men holding down responsible posts, there's always room for more. **We shall be pleased to hear from *anyone* who would like to discuss the possibilities of joining Truscon in 1960.**

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